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IFICATION OF CANDIDATE ZERO MAINTENANCE PAVING MATERIALS

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Offices of Research & Development

Washington, D. C. 20590

FOREWORD

This report presents the findings of a State-of-the-Art review in which available information on a broad spectrum of materials which may have potential for improving the characteristics of pavements with regard to reducing future maintenance, was synthesized. Materials studied included Gussasphalt, asbestos asphalts, sulfur modified asphalts, noncalcareous inorganic cements, expansive cements, fiber reinforced concrete, polymers in concrete, sealants, ceramics, prestressed concrete and vacuum processed concrete. None of the materials or materials systems will singularly provide the desired improved performance. However, many of the materials possess desirable characteristics and when combined with other materials or techniques could greatly extend the maintenance free life of a high traffic volume pavement.

This report is being distributed to materials and pavement researchers involved in work with the materials studied.

Charles F. Scheffey

Director, Office of Research

NOTICE

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16. Abstract

A broad spectrum of materials which may have potential for improving the characteristics of pavements with regard to reducing future maintenance were reviewed with those materials or materials systems possessing the most advantages being considered as candidates for a zero maintenance paving system. These included Gussasphalt, asbestos asphalts, sulfur modified asphalts, noncalcareous inorganic cements (sulfur and phosphate), expansive cements, fiber reinforced concrete, polymers in concretes, sealants, ceramics, prestressed concrete and vacuum processed concrete. These materials were compared to conventional asphalt and portland cement concrete systems on the basis of mechanical properties, ease of construction and maintenance, environmental compatibility, availability, cost, and possible uses. None of the materials or materials systems studied will provide a singular answer to the problem of zero maintenance. However, many of the materials possess characteristics which could be incorporated into pavements to greatly extend the period prior to maintenance.

The other volume of this study is:
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PREFACE

This report was prepared for the Federal Highway Administration (FHWA), Office of Research and Development, Materials Division, Department of Transportation, Washington, DC, under an intra-government contract, Order No. 4-1-0189, dated 3 May 1974. The FHWA technical monitor for this work was Mr. K. C. Clear.

This report was compiled and organized by Mr. G. C. Hoff, Chief, Materials Properties Branch, Concrete Laboratory (CL), US Army Engineer Waterways Experiment Station (WES). The various chapters within the report were prepared by the following WES staff members:

Chapter No.	
1, 7, 8, 9, 10, 15	George C. Hoff, Chief, Materials Properties, Branch, Concrete Laboratory
2, 3, 4, 5	Lenford N. Godwin, Civil Engineer, Pavement Materials Research Facility, Soils and Pavements Laboratory
6	Alan D. Buck, Acting Chief, Petrography and X-Ray Branch, Concrete Laboratory
11	Tony B. Husbands, Chief, Chemistry and Plastics Branch, Concrete Laboratory
12	Katharine Mather, Chief, Engineering Sciences Division, Concrete Laboratory
13, 14	Kenneth L. Saucier, Research Civil Engineer, Concrete and Rock Properties Branch, Concrete Laboratory

The report preparation was done under the general supervision of Messrs. B. Mather, Chief, CL; J. M. Scanlon, Jr., Chief, Engineering Mechanics Division, CL; J. P. Sale, Chief, Soils and Pavements Laboratory (S&PL); A. H. Joseph, Chief, Pavement Investigation Division, S&PL; and T. D. White, Chief, Pavement Materials Research Facility, S&PL.

Directors of WES during this study and preparation of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
square inches	6.4516	square centimetres
miles	1609.3	metres
square feet	0.09290304	square metres
square yards	0.8361274	square metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
cubic centimetres	0.000001	cubic metres
inches per minute	25.4	millimetres per minute
centimetre per second	10	millimetres per second
grams	0.001	kilograms
pounds (mass)	0.4535924	kilograms
tons (2000 pounds mass)	907.1847	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic yard	0.59327638	kilograms per cubic metre
pounds (force)	4.448222	newtons
pounds (force) per inch	175.1268	newton per metre
pounds (force) per square inch	0.006894757	megapascals
foot-pounds (force) per inch	53.378661	joule per metre
foot pounds (force) per square foot	14.59390	joule per square metrè
Btu inch per hour square foot degree Fahrenheit	0.1442279	watt per metre kelvin
poise	0.1	pascal · second
centipoise	0.001	pascal second
centistokes	0.000001	square metres per second
milliliter	0.000001	cubic metres

Multiply	By	To Obtain
ounce (US liquid)	0.00002957353	cubic metres
ohms per square foot	10.76391	ohms per square metre
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[a]

[a] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use $K = (5/9)(F - 32) + 273.15$.

CHAPTER 9

FIBER REINFORCED CONCRETE

INTRODUCTION

9.1 BACKGROUND

The concept of fiber reinforcement for materials weak in tension is almost as old as recorded history. Patents on the use of reinforcing elements in cement matrices can be found as early as 1874[57] with other patents developing through the 1920's.[159, 185, 280, 345, 357, 513] Although asbestos cement products have been in production for more than 75 years,[283] it is only within the last two decades that any substantial development of other types of fiber reinforced cements and concretes has been undertaken. This has been prompted by both the availability of economical fibers and the need to extend the use of cheaper construction materials such as concrete into other applications than it is normally used for.

When considering improvements in concrete, the tendency is to think in terms of strength alone. The inclusion of fibers as reinforcement is generally thought of as a panacea for the poor tensile strength of cement and concrete. The role of fiber reinforcement, however, is not so much in the improvement of static strength as it is to control cracking. The controlled cracking results in improved ductility, energy absorption, and resistance to impact, shock, and thermal loading of the composite. The ability to control the size and amounts of cracks will also lead to improved durability as composites can be designed to have the ability to reduce the amount of intrusion by aggressive environments.

This chapter of the report is concerned with developments in fiber-reinforced cementitious materials through 1975. For highway applications, the specific interest would be mostly in concrete, but much of the work on fiber-reinforced cements and mortars is applicable to the understanding of fiber-reinforced materials behavior and is included in the review. No attempt is made to explore the rheological characteristics of fiber-reinforced cementitious materials or the techniques for producing and placing these materials. This information is contained in the many references on this subject, however. The bibliography on fiber-reinforced cementitious material contained in this chapter is the most comprehensive developed to date on this subject, but because of its size, it was impossible to use all of the references in the development of this review. Emphasis is placed, however, on the types of fibers, the reinforcing mechanism, mechanical properties, and highway applications. Notations used in this chapter is described in Appendix A.

9.2 DESCRIPTION OF MATERIAL

9.2.1 Fiber-Reinforced Concrete. Fiber-reinforced concrete is defined as concrete made of hydraulic cements containing fine or fine and coarse aggregate and discontinuous, discrete fibers.[5]. Continuous meshes, woven fabrics, and long rods are not considered to be discrete fiber type reinforcing elements. A very wide range of discontinuous, discrete fiber types is available for use as reinforcement in a variety of different cement-based matrices.

9.2.2 Fibers. Many different types of materials have been investigated and used as reinforcing fibers in cements, mortars, and concretes, with the most common being steel, glass, polymeric fibers, and asbestos. Other fibers such as mineral wool and vegetable fibers have also been used. Carbon fibers, a recent development, have also been proposed for use in cement and concrete. The following paragraphs highlight some of the characteristics of these fibers. Table 9.1 is a summary of typical fiber properties.

9.2.2.1 Steel Fibers. Steel fibers in concrete have been extensively studied and used particularly in the construction of pavements and overlays. Steel fiber manufacturing possibilities include broaching or milling of stacked sheet and strip, rotary and reciprocal cutting or shearing of wire, sheet or strip, and direct conversion of molten steel to discontinuous fiber (hot-melt spinning).[512] At present, most steel fibers are produced by cutting round drawn wire to length and by slitting and chopping light flat rolled sheet or strip. The hot-melt process has been principally applied to stainless steel fibers, but some fibers made from melted scrap metal are being produced for concrete. The drawn wire fiber may be smooth, flattened, or deformed. The flat, slit and chopped fiber is straight but evidences varying degrees of twist and camber which result from the manufacturing process. The hot-melt fibers are distinctly irregular in cross-section.

Steel fibers are available in various lengths and diameters (or equivalent diameters for rectangular fibers). Most are sized to have aspect ratios (length divided by diameter) from 30 to 150 for fiber lengths of 1/2 to 2-1/2 in. Two popular forms of steel fiber have been used in the United States in the past. The first is a crimped wire produced by flattening evenly spaced portions of a round wire. This fiber has typically been 0.016 in. in diameter and 3/4 in. long.[618,639] The second is a flat fiber produced by shearing sheets or flattening wires to result in a fiber that is typically 0.010 in. by 0.022 in. by 1 in. long.[512,598] The trend in concrete work recently, however, has been to slightly longer fibers than 3/4 to 1 in. and these are available from the same producers of the shorter fibers. A recent European introduction to the United States market has been a drawn wire fiber provided with hooked ends and with the fibers glued together in small units with a water soluble glue. The longer, stiffer units of fibers make fiber introduction and initial mixing easier, and once in the mixer and during mixing, the glue dissolves and the fibers separate into single filaments to provide their reinforcing action. It is available in aspects ratios from 85 to 125 with fiber lengths to 2 in. United States production of this fiber is planned for 1977.

Most steel fibers are made from plain carbon steel but fibers with higher tensile strengths are also available. The hot-melt fibers made from scrap metal tend to be slightly lower in strength and elongation. Stainless steel fibers for refractory concrete are available with tensile strengths ranging from 290,000 psi to 363,000 psi. The tensile strength of the fiber is not critical if the composite fails by fiber pull-out, bur if fiber yielding occurs, then fiber strength may be decisive in determining the ultimate strength capability of the composite.[555]

The bond developed between the fiber and matrix is influenced by both the geometry and surface condition of the fiber. Surface indentations and different shapes appear to impart only limited increases in strength properties but do significantly affect the post cracking behavior. Numerous coatings have been applied to steel fibers to improve bond. Table 9.2 shows the results of both chemical and mechanical treatments on the force required to pull out a wire from a test specimen. Galvanizing produced the most improvement of the chemical treatments, but the greatest improvements came from mechanical treatments.[242]

The environmental durability of steel fibers (except stainless steel) is variable. It will rust in the presence of water and air but can be protected from this by the cement paste. It has good resistance to alkalis and performs poorly in the presence of many acids. Stainless steel fibers have a generally good environmental durability.

9.2.2.2 Glass Fibers. Glass fiber, being man-made, has no limits with respect to its form and fabrication. It is produced by attenuating molten glass by various techniques which include: blowing compressed air or steam at a stream of molten glass; centrifuging; a continuous filament method; and mechanical drawing. Mechanical drawing of continuous filaments provides the most convenient and attractive method of glass fiber manufacture.[146] Not all glasses can be fiberized this way, however. The filaments formed by these various processes are combined to make strands, rovings, woven or chopped strand mat, or in the case of blowing, are left as glass wool. A typical glass fiber strand can have from 200 to 400 filaments in it with each fimalent being 0.0035 in. in diameter and a roving can have 20 to 60 such strands.[198]

In general, glass fibers fall into two groups: soda-lime-silica glass referred to as A-glass; and boro-silicate glass referred to as E-glass. More recently a glass containing zirconia (ZrO_2) has been developed and given the designation of AR-glass or alkali-resistant glass. Type E and AR glass are more commonly used in cement composites. The tensile strength and elastic modulus of the glasses varies from

300,000 to 500,000 psi and 10-12 × 10^6 psi, respectively. Elongations of 2 to 3.5 percent are possible.[146] Glass fibers will tolerate temperatures to 1500 F with little harmful effect. Numerous investigators* have reported on the degradation of E-glass figers in the high alkaline environment of a cement composite. AR-glass fibers are more stable than E-glass in portland cement pastes but still undergo a slight reduction in strength with time.[106,333] E-glass fibers have been successfully used in gypsum products.[326,334,495] The strength reduction of AR-glass fibers can be prevented by adding fly ash to the portland cement.[332]

9.2.2.3 Polymeric Fibers. Fibers can be produced from a large number of synthetic polymeric materials. Most are very expensive, however, and many are not compatible with a portland-cement environment. The three fibers most commonly mentioned as reinforcement of cement and concrete are polypropylene, nylon, and polyesters. Other fibers such as rayon,[184] polyethylene,[184] Saran,[183,184,272] cellulose acetate,[183,184] Orlon,[183,184] and polyvinly alcohol[371,578] have also been investigated. Polypropylene is the most commonly used polymeric fiber in cementitious systems.** It is produced as both a filament fiber and as a fibrillated twine. The filament form is produced by melting the polymer and drawing it through sized dies. The fibrillated twine is produced from extruded film which is cut into tapes approximately 23/32 in. wide. The tapes are then heated and stretched to approximately eight times their original length. They are then fibrillated by passing over angled rollers and twisted into a twine at approximately 12 turns per foot.[146] Normally the adhesion of cement to polypropylene is very poor as is evidenced by the fact that polypropylene sheeting is sometimes used as a bond breaker in joints in concrete construction. The bond to filament fibers is also poor although regular constrictions of one type or another will improve the mechanical bonding. The twisted, irregular surfaces of fibrillated polypropylene provides an excellent surface for

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* References: 65, 106, 198, 293, 333, 334, 490, 602

** References: 9, 94, 110, 117, 118, 140, 168, 358, 449, 657 the intrusion of cement paste thus significantly improving the inbedment of the fiber in the matrix. Typical single filament and fibrillated fiber diameters are greater than 0.0016 in.[146] The tensile strengths of the single filament and fibrillated fibers have been reported as 93,000 and 57,000 psi, respectively, while the elastic moduli were 0.7×10^6 and 1.1×10^6 psi, respectively.[146] The single filaments elongate as much as 18 percent while the fibrillated twine only reaches 8 percent. There is a progressive loss of strength in both fiber types with increasing temperature. This has been reported as a 50 percent loss at 212F. The polypropylene has a good but not exceptional resistance to water, HCl, and alkalis.[146]

Nylon fibers are produced by the melting and drawing process and are thus only available in the monofilament form. Nylon twine, formed by twisting several monofilaments together, has also been cut into small lengths and used in concrete. Nylon's strength, chemical inertness, durability, and elongated temperature resistance are no less than in polypropylene but it is more expensive than polypropylene.[658] The tensile strength of nylon fiber has been reported from 60,000[367] to 130,000 psi[658] with elastic moduli from 350,000[367] to 600,000 psi.[146,658] Elongations of 13 to 25 percent are possible.[146,367] Typical monofilament diameters are in excess of 0.0016 in.

Polyester fibers (Terylene and Dacron) have reasonably good tensile strengths (127,000 psi) and a somewhat higher elastic modulus (1.6×10^6 psi) than either nylon or polypropylene.[146] Polyester fibers, along with rayon fibers are adversely affected by alkalis, however,[146,184;] and do not perform well with time in cementitious systems. The strength loss of polyester fibers with increasing temperature is comparable to polypropylene.[146] These fibers have also been reported as having fiber diameters greater than 0.0016 in. and elongations of 10 to 11 percent.[146]

All of the above polymeric fibers have an elastic modulus less than that of concrete and as such are not expected to make any substantial contribution to the strength of a fiber reinforced concrete. They should significantly improve post-cracking behavior and impact resistance however Majumdar[331] called attention to a recently developed high modulus organic fiber called PRD-49 (Kelvar) and suggested that it may be useful as a reinforcement in cement composites. PRD-49 has tensile strengths from 400,000 to 430,000 psi and elastic modulus from $12\text{-}19 \times 10^6$ psi.[369] Elongations at break are 2.0 to 3.3 percent. Fiber diameters from 0.005 to 0.007 in. were reported.[369] No information on the use of this fiber in cement or concrete could be located, however.

9.2.2.4 Crystalline Silicate Fibers (Asbestos). Certain silicate materials such as wallastonite and asbestos occur naturally in a fibrous form but only asbestos has been used extensively as reinforcement. Chrysotile ($3\ MgO \cdot SiO_2 \cdot 2\ H_2O$) or white asbestos is derived from the serpentine group of minerals and constitutes 90 percent of the world's asbestos reserve. Crocidolite ($Na_2O \cdot Fe_2O_3 \cdot 3\ FeO \cdot 8\ SiO_2 \cdot H_2O$) or blue asbestos is derived from the amphibole group of minerals and is the strongest and stiffest of the five asbestiform mineral species which exist in the amphibole group. Chrysotile is a white silky fiber which can have a diameter as small as 0.4μ in. whereas crocidolite cannot be split down much less than 4μ in. in diameter. Both fibers can occur in lengths up to 4 in. but lengths of 1 in. or less are more common.

Asbestos fibers are the most commonly used reinforcement for fiber-reinforced cement materials.[288] They have a high chemical resistance and good mechanical properties with tensile strengths of 72,000 psi[288] to 500,000 psi[146] and a modulus of elasticity of 12×10^6 psi[5] to 28×10^6 psi.[146] Elongations at break of 3 percent or greater have been reported.[146] With high temperatures all varieties of asbestos fibers break down throuth a series of internal reactions which may begin as low as 400°F. These reactions are very rapid in the range of 1100 to 1800°F.[146] The asbestos is not affected by water and is attacked only slightly by alkalis. The resistance to HCl is better for crocidolite asbestos than for chrysotile asbestos.

The asbestos content in asbestos-cement products can vary from 8 to 70 percent[658] but 8 to 16 percent is more common.[367] The only reported information on asbestos-concrete[337*] used 0.5 to 11 percent asbestos by weight of the cement although the higher amounts were possible only by using post-mixing water suction processes. The asbestos fibers can withstand very heavy mechanical treatment and are less prone to being affected by the mixing action in concrete.[288,658]

9.2.2.5 Carbon Fibers. Carbon fibers are a recent development which is finding some interest as fiber reinforcement in portland cement systems.[14,73,163,499,615,616,617] Carbon fibers are produced by the carbonization of suitable organic fibers and can be made in either short or continuous lengths. In both an American and British process, an organic textile fiber is converted at very high temperatures into a carbon fiber in which small graphite crystallites are aligned in a particular way by the application of tension during the manufacturing process. The American process begins with certain rayon fibers while the British process uses polyacrilonitrite fibers. A similar Japanese process uses pitch as the basic material. The diameter of single carbon filaments has been reported as 0.0003 in.[146,615] with tows of 10,000 filaments of almost unlimited length being produced.[615] Each tow of carbon fiber is opened out to form a layer of single filaments which is then fixed in a water soluble medium to form a tape of approximately 3-in. width. Multiple tows can be opened out simultaneously to produce a tape of almost any width.[615] Both the strength and elastic modulus of carbon fibers vary within wide limits but can be controlled by regulating the degree of stretch applied to the fiber.[146] Elastic moduli of $35\text{-}65 \times 10^6$ psi and tensile strengths of 200,000 to 450,000 psi have been reported.[146,615] It has been suggested that carbon fibers would be most effective in cement composites when used in continuous lengths parallel to the direction of stress.[447,615]

9.2.2.6 Vegetable Fibers. Fibers of vegetable origin are all based on cellulose and form an almost inexhaustible, reproducible supply throughout the world. Unfortunately, their performance as a reinforcement is poor as they are the weaker of the families of fibers, are

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* Reference 242, pp 42-44, discussion by Dave.

susceptible to microbiological attack and rotting, and degrade in an alkaline environment.[146,288] Vegetable fibers are usually classified by the part of the plant from which they come, that is seed and fruit fibers, leaf fibers, and stem fibers.[146]

The most important seed and fruit fiber is cotton. Cotton has a typical fiber diameter of 0.0004 and 0.0008 in. and lengths up to 2-1/2 in.[146] Its tensile strength and modulus of elasticity vary from 40,000 to 120,000 psi and 0.8 to 1.6×10^6 psi, respectively, with elongations being 5 to 10 percent. Cotton has a poor environmental durability and does not perform well in the presence of water, acid, or alkalies. At increasing temperatures it loses strength at 300°F and decomposes between 365 to 395°F.[146]

The best known leaf fiber is sisal. It has a typical diameter of 0.0003 to 0.0019 in. and has been produced in lengths to 48 in. Sisal has a tensile strength of 120,000 psi with elongations up to 3 percent. Its behavior at room temperature is basically brittle. It tends to lose strength with time when held at higher temperatures and loses all strength at 300°F. It is highly susceptible to water and acid.[146]

Stem fibers come from the fibrous bundles in the inner bark of the plant stem and include jute, flax, and hemp. Hemp has typical fiber diameters from 0.0006 to 0.0020 in. and lengths to 72 in. Its tensile strength is very low for fibers being only 55,000 psi with elongations up to 1.8 percent.[146] The behavior of hemp is very much like sisal with respect to strength loss at higher temperatures and environmental durability. Jute has been reported as maintaining impact resistance in concrete up to 90 days.[538]

<u>9.2.2.6 Glass and Ceramic Wool Fibers</u>. Glass wools, often referred to as rock wool or slag wool, are made from rocks or metallurgical slags by blowing compressed air or steam at a stream of the molten rock or slag or by spinning.[287] Ceramic wools consist of alumino-silicate fibers and are made by blowing alumino-silicate glasses.[146] Although these fibers are mostly glassy in structure, they are classified as ceramics and special furnaces are required in their production because of their very high melting points. Rock wool normally has single

filament diameters from 0.00008 in. to 0.00020 in. with lengths varying from only a few times the filament diameter to 2 in. The distribution of different fiber lengths within the wool is not known.[287] The ceramic wools have filament diameters approaching 0.00011 in. and lengths up to 10 in.[146] The tensile strength, elastic modulus, and elongation of rock wool fibers have been reported[287] as 70,000 to 110,000 psi, $10\text{-}17 \times 10^6$ psi, and approximately 0.6 percent, respectively. No property data were found for ceramic wools. The environmental durability of the wools is expected to be fair to poor.[146]

9.2.3 Matrices. The three basic matrices considered for fiber reinforced cement and concrete are portland cement, high alumina cement, and gypsum plaster. Besides ordinary portland cement, other special portland cements such as expansive cement,[355] supersulphated cement,[198] and regulated-set cement[200,206,207,208,428,429] have also been reinforced with fibers. High alumina cement[99,175,198,306,432] and gypsum cement[13,146,178,334,335,349,388,495] are usually used with E-glass fibers to minimize the effects of alkalies on the glass. Combinations of portland cement and pozzolans have also been recommended for steel fibers to improve workability[274,277] and for glass fibers to reduce the activity of alkali attack on the fibers.[332,541] Only limited success was achieved in the later instance.

FIBER REINFORCING MECHANISM

The properties of fiber-reinforced mortars and concretes are influenced by many paramenters. With regard to the fibers these include: the fiber geometry (usually described by the aspect ratio, ℓ/d where ℓ = fiber length and d = fiber diameter); the volume fraction of the fibers in the mixture, V_f, and the orientation of the fibers in the mixture. The crimping or bending of fibers is another geometry consideration. The same variables that affect conventional concrete such as water-cement ratio, air content, density, etc., also have a significant effect on the properties. As the benefits of fiber additions are dependent on the development of bond between the fibers and the matrix, the variables which affect bond should be given special consideration.

Two distinct stages of behavior are evident in the load deformation curve of fiber-reinforced concrete tested in flexure (Figure 9.1). The curve in Figure 9.1 is more or less linear up to some point which will be described as the "proportional limit." Beyond that point the curve is distinctly non-linear reaching a maximum of ultimate strength or load before beginning a descending portion. The proportional limit is often referred to as the "first crack strength." The region beyond that point is described as "post-cracking" behavior.

Two mechanisms for prediction the first crack strength or the proportional limit of fiber-reinforced concrete have been proposed. These are the fiber spacing concept[485] and two-phase composite materials approach.[521] The fiber spacing mechanism relates the first crack strength to the spacing of the fiber reinforcement. The composite materials mechanism relates the proportional limit to the volume, orientation, efficiency, and aspect ratio of the fibers. A state-of-the-art report by the American Concrete Institute (ACI)[5] concluded that it is generally agreed that the ultimate strength of fiber-reinforced concrete is relatively insensitive to fiber spacing and depends primarily on the volume, aspect ratio, and bond characteristics of the fibers.

9.3 SPACING CONCEPT

In developing the spacing concept, Romualdi and Batson[485] used both the ability of the fibers to act as crack arrestors and a linear elastic fracture mechanics approach. They attempted to increase the tensile strength of mortar by decreasing the stress intensity factor using closely spaced wires as crack arrestors. Romualdi and Mandel[487] extended this work and using an assumption that only 41 percent of the total amount of fiber reinforcement was effective in crack control, developed a "spacing" parameter of the form

$$\text{spacing} = 13.8\ d\sqrt{\frac{1}{p}} \tag{9.1}$$

where p is the volume fraction of fibers expressed in percent. Equation 9.1 defines the geometrical arrangement of fibers in matrix.

Other variations of this spacing parameter have also been proposed. McKee,[355] used the spacing of centroids with fibers at random angles to derive the expression

$$\text{spacing} = \sqrt[3]{\frac{100\ V}{p}} \tag{9.2}$$

where V is the volume of one fiber and p is the volume percentage of fiber in the mortar. Swamy and Mangat,[568] however argued that when fibers are randomly oriented and distributed throughout the matrix, the use of a spacing concept solely on an assumed geometrical arrangement as in Equations 9.1 and 9.2 loses its significance. They considered both the interfacial shear stress distribution due to load transfer from the matrix to the fiber and the local discontinuities in bond stress distribution and derived a new "effective spacing" equation given as

$$\text{effective spacing} = 0.984\sqrt{\frac{1}{p}\cdot\frac{d}{\ell}} \tag{9.3}$$

Parimi and Rao[422,423] expressed the spacing formula as

$$\text{spacing} = 8.86\,d\sqrt{\frac{1}{\beta p}} \tag{9.4}$$

where β is the factor which, when multiplied by the length of a fiber, results in its effective length in the direction perpendicular to the cross section. Using probability concepts, they concluded that $\beta = 0.637$ as opposed to $\beta = 0.405$ which it would have to be to produce the derivation shown in Equation 9.1. The resulting expression for spacing was

$$\text{spacing} = 11.1\,d\sqrt{\frac{1}{p}} \tag{9.5}$$

For comparison purposes, the four equations are plotted in Figure 9.2 for a 0.016-in. diameter by 1.0-in. long fiber used at various volume percentages. It should be noted that the term p in Equations 9.1 through 9.5 is expressed as actual percent.

9.4 COMPOSITE MATERIALS CONCEPT

Various rigorous theoretical treatments of the behaviorial mechanics of fiber-reinforced materials have evolved over the years and attempts have been made to apply these to fiber-reinforced concrete. The most simple case to consider is that where the fibers are aligned paralled to the applied stress. Using the "principle of mixtures," which states that the properties of the composite are proportional to the volume fractions of the constituents, the elastic modulus of the composite in direct tension can be given as

$$E_c = E_f V_f + E_m \left(1-V_f\right) \tag{9.6}$$

where E = modulus of elasticity and the subscripts c, f, and m denote the composite, fiber and matrix respectively. Equation 9.6 assumes that the continuous fibers are bonded to the matrix in an ideal manner and that the overall strain in the composite is equal to the strain in the fibers and the matrix. The Poisson's ratio for the fiber and matrix are assumed equal and are neglected in this derivation.

Of course, in actual fiber-reinforced cement or concrete composites, the fibers are generally not continuous or aligned in a direction parallel to the applied stress or bonded to the matrix in an ideal manner, nor is the Poission's ratio of fiber and matrix the same. To compensate for these deficiencies various forms of Equation 9.6 have been proposed.

Krenchel[287] included an efficiency factor for reinforcement of arbitrary orientation, λ, and modified Equation 9.6 to the form

$$E_c = E_f V_f \lambda + E_m \left(1-V_f\right) \tag{9.7}$$

In Krenchel's[287] actual equation, V_f was replaced by another term which represented the area fibers represented on an arbitrary plane. λ varies from $\lambda = 1$ for fibers paralled to the force to $\lambda = 0$ for fibers perpendicular to the force. For three dimensional reinforcement uniformly distributed over all directions, he concluded $\lambda = 1/5$. He further concluded that while the initial modulus of elasticity depends on λ, the curvature of the stress-strain diagram and the ultimate strength of the test specimen would depend on a "total efficiency factor," λ', where $\lambda > \lambda'$ because λ' was the product of λ and a further reducing factor which took into account the ratio length of fiber and the necessary anchorage length of that fiber. Equation 9.7 can also be written in terms of modular ratio, n, where

$$n = E_f/E_m$$

or

$$\frac{E_c}{E_m} = 1 + V_f(n\,\lambda - 1) \tag{9.8}$$

This relation shows the sensitivity to both volume fraction of fibers and the modular ratio thus indicating their relative importance.

Pakotiprapha, et al,[418] modified Equation 9.7 even further by including a correction factor, Ψ[661], to account for the stress distribution at the end portion of the fibers. This is analagous to Krenchel's reducing factor in the total efficiency factor. The equation form is then

$$E_c = E_f\, V_f\, \lambda\, \Psi + E_m \left(1-V_f\right) \tag{9.9}$$

For the lengths of wires used in their work,[418] the average correction factor, Ψ, varied between 0.52 and 0.64.

The efficiency factor, λ, has been reported to vary from 0.17 to 0.80.[5] In most cases, little experimental data are available to substantiate what the actual values may be. They may, in fact, vary from concrete to concrete depending on the volume fraction of fibers, the size of specimen, the mode of consolidation, and the mixture proportions.

Equation 9.9 is certainly valid for fiber-reinforced concrete in its elastic range. Shah and Rangan[521] have shown however that the composite materials approach could be extended to describe the entire behavior of the concrete until failure. They predicted the behavior of fiber-reinforced concrete prisms subjected to uniaxial compression by dividing the stress-strain curve and Poisson's ratio-strain curve of concrete into several segments. The assumption was made that the concrete behaved elastically within each segment. By determining the secant modulus of elasticity (E) and Poisson's ratio (μ) for each segment and assuming that the fibers always behave elastically with elastic constants, E_f and μ_f, they calculated the elastic modulus of each composite section and reconstructed the stress-strain curve. A good comparison between calculated and experimental curves was obtained and qualitatively demonstrated the plausibility of applying the composite materials approach to complete concrete behavior.

9.5 POST-CRACKING BEHAVIOR

Once the matrix cracks, the stress carried by it is thrown onto the fibers. From that point on, the maximum load is controlled primarily by fibers gradually debonding and pulling out. The point at which this begins has been described as the "first crack strength" of the composite and is defined for fiber-reinforced concrete as the proportional limit of the concrete or in other words, that point below which the concrete is essentially linearly elastic.[5] During this pulling out process by the fibers, the stress in the fiber is substantially less than the yield stress of the fiber even up to the maximum load. For long, aligned fibers, pullout would not occur due to the large fiber lengths and successive debonding would result in fracture of the fibers. When this occurs, the strength of the composite is determined only by the fracture strength of the fibers and the amount of fibers in the mixture:[420]

$$\sigma_{fc} = \sigma_{fu} V_f \tag{9.10}$$

This is the limit case and represents the lower strength boundary of real composites beyond a critical volume fraction of fibers.[285]

For discontinuous aligned fibers the strengths of composites are lower than those determined by Equation 9:10 due to the probabilstic dispersion of the centroids of the fibers resulting in the weakest link and also due to the steep stress gradient in the fibers.[661] This reduction is accomplished analytically by using an average fiber stress in lieu of the maximum fiber stress where the average fiber stress is equal to the product of the maximum fiber stress and the efficiency factor, Ψ . For a linear fiber stress distribution which results from a uniform bond stress distribution, which in turn normally occurs after debonding, the efficiency factor, Ψ, is expressed as

$$\Psi = \left(1 - \frac{\ell_c}{2\ell}\right) \tag{9.11}$$

where ℓ_c is the critical fiber length and ℓ is the actual fiber length. From Equation 9.11 it can be seen that when the actual fiber length is critical $(\ell = \ell_c)$, the composite is only 50 percent efficient.

When the fibers are both discontinuous and random, Equation 9.10 must be further modified by a simple orientation factor, λ , which will further reduce the strength of the composite. Equation 9.10 can then be written as

$$\sigma_{fc} = \sigma_{fu} \Psi \lambda V_f \qquad (9.12)$$

which is the lower boundary case for maximum strength of a composite containing random discontinuous fibers. This expression suggests that increasing the volume percentage of fibers linearly increases the strength of the composite. Monfore,[367] in a review of fiber reinforcement of cement paste, mortar, and concrete, found this to be more or less the case.

The limit case which represents the upper strength boundary can be developed from the principle of mixtures previously applied to the elasticity of the composite. For equal elastic strains in both the fibers and matrix, the direct tensile strength, σ_{ct}, of a composite containing uniaxial continuous fibers can be expressed as

$$\sigma_{ct} = \sigma_{mt} \left(1-V_f\right) + \sigma_f V_f \qquad (9.13)$$

where the suffix, τ, represents direct tension. Applying the same reasoning as before to fiber efficiency and orientation, Equation 9.13 can be written as

$$\sigma_{ct} = \sigma_{mt} \left(1-V_f\right) + \sigma_f \lambda \Psi V_f \qquad (9.14)$$

Swamy, et al,[570] considered the load transfer from matrix to fiber by interfacial shear stress, τ, the tensile stress in the fiber, and the fiber stress distribution and determined the critical fiber length, ℓ_c, needed for the fracture stress to be reached:

$$\ell_c = \sigma_{fu} \cdot \frac{d}{2\tau} \qquad (9.15)$$

This derivation is also shown by Pakotiprapha, et al.[418] Other derivations for critical length can be found in the literature. Combining Equations 9.14 and 9.15 the composite strength can be written to include the aspect ratio of the fiber, as

$$\sigma_{ct} = \sigma_{mt}\left(1-V_f\right) + 2\tau\,\lambda\,\Psi\,V_f\left(\frac{\ell c}{d}\right) \tag{9.16}$$

Swamy, et al,[570] have used the simple linear relationship between modulus of rupture of the matrix, σ_{mr}, and its direct tensile strength, σ_{mt}, where $\sigma_{mr} = \delta \cdot \sigma_{mt}$, plus a factor, $\propto$, which relates the extreme fiber stress, the elastic section modulus, and the applied moment, to modify Equation 9.16 so as to make it applicable for flexual strengths which are useful for fiber reinforcement of concretes. The resulting expression

$$\sigma_{cr} = \frac{\propto}{\delta}\,\sigma_{mr}\left(1-V_f\right) + 2\propto\tau\,\lambda\,\Psi\,V_f\left(\frac{\ell_c}{d}\right) \tag{9.17}$$

was generalized into the form

$$\sigma_{cr} = A\sigma_{mr}\left(1-V_f\right) + B\,V_f\left(\frac{\ell}{d}\right) \tag{9.18}$$

where the constants A and B can be determined by a plot of composite strength versus $V_f\left(\frac{\ell}{d}\right)$.

The term $A\sigma_{mr}\left(1-V_f\right)$ represents the contribution of the matrix at maximum load. The maximum value of the constant A is unity. The term $B\,V_f\left(\frac{\ell}{d}\right)$ represents the contribution of the fiber and is influenced by both volume fraction of fibers and their aspect ratios. The constant B depends on fiber efficiency and orientation, and the bond strength between the fibers and the matrix. When any of these improve, so does the strength contribution by the fibers. It should be noted that Equation 9.18 only applies when failure occurs from debonding of the fibers. Values for A and B have been shown to be 0.843 and 2.93, respectively, for first crack flexural strength of steel fibrous concrete and 0.97 and 3.41, respectively, for the ultimate strength of the same concrete.[570]

9.6 FLEXURAL STRENGTH

The precise influence of randomly distributed fibers on the flexural strength of mortars and concrete is often clouded because of the many other variables both within and outside the matrix which also influence the flexural strength. A reasonable description of the behavior of a fiber-reinforced concrete beam is that a fiber-reinforced beam subjected to bending by increasing load will, like any other nonreinforced concrete beam, develop strains in the outer tensile region of the beam. When a critical strain is reached, the beam cracks but unlike the nonreinforced beam, the cracks do not propagate through the beam but are arrested by the fibers which span the cracks (Figure 9.3). A portion of the load is maintained across the crack by the portion of the fiber which is imbedded in the matrix, thus maintaining equilibrium in the beam. The ability of the fiber to maintain this load is affected by the bond developed between the fiber and the matrix, the imbedment length of the fiber, the strength of the fiber, and the level of loading.[460] Due to the crack formation, the measured tensile strains are increased and hence the neutral axis moves up into the top half of the beam to a new location.[418] With further load, the measured tensile strains increase at a greater rate than the compressive strains. The location of the neutral axis continues to change until there is no simple relationship between the measured strain and apparent stress sustained across the crack. Plastic stress blocks which have been developing within the system then influence the beam behavior.[418] The fibers themselves begin to lose their effectiveness by either gradual debonding and pullout or if they are long enough to maintain an effective bond, they will fail by yielding and fracture.[420]

<u>9.6.1 Effect of Steel Fibers</u>. When a beam containing high strength, high modulus fibers behaves in the above manner, intuitively the ultimate flexural steesses and strains would be expected to be greater than those of a nonreinforced beam. The first crack strength

is a different case however. If any influences of strengthening throughout the beam by the fibers, are neglected, the matrix should crack at the same strain that it would if the beam were nonreinforced. Hence, the first crack strength should not be influenced by the addition of fibers. Little or no change in the first crack strength of steel fiber reinforced concrete has been reported.[380,418,519,521,618] Substantial improvements have also been reported, however,[304,505,534,565,569] thus suggesting a more complex fiber strengthening mechanism throughout the beam. These discrepancies have been attributed to variations in specimen preparation and testing techniques and also in the definition of a "first crack" (see Figure 9.1). In many instances where improvements have been noted, the flexural strength of steel fiber reinforced beams has been related to numerous parameters, all of which include the term $V_f(\ell/d)$.[261,304,565,568] This parameter includes the principal variables of the fiber itself, that is, amount, length, and diameter but does not consider other factors which may also influence the cracking strength and behavior such as specimen size, mixture proportions, cement differences,[261] surface condition of the fiber, fiber alignment, consolidation of the mixture, uniformity of fiber distribution,[534] and presence of aggregate.[568] Hannant and Spring[214] showed that by using a fiber aligning technique, the flexural strength of a mixture containing 2 percent steel fiber was improved 2.5 times over beams containing a random distribution of the same fibers. Edgington and Hannant,[133] using external vibration and casting flexural specimens both horizontally and vertically, found that the influence of external vibration on the horizontal casting produced strengths 50 to 100 percent better than those of specimens cast vertically due to preferential alignment of the steel fibers perpendicular to the direction of loading in the horizontally cast beams.

Numerous investigators have shown that a definite relationship exists between ultimate flexural strength and steel fiber aspect ratio. For a given mixture and volume of fibers this relationship is generally linear with ultimate flexural strength increasing with increasing aspect ratio.[505,521,567] It has also been shown to be curvilinear.[261] There appears to be, however, no singularly unique relationship between ultimate flexural strength and fiber aspect ratio.[567] Swamy and Mangat[567] concluded that for any given fiber diameter and volume of fibers, there is a linear relation where the ultimate flexural strength increases with increasing aspect ratio until "curling up" of the fibers occurs and causes a reduction in strength. Data of Shah and Rangan[521] also show this strength reduction once a certain aspect ratio is reached. The limiting steel fiber aspect ratios for "curling up" are reported to be in the range of 100[521,567] to 150.[484] Swamy and Mangat[567] also described a "transition aspect ratio" for two given diameters of steel fibers. Below an aspect ratio of 70, thinner fibers appear to be more effective than thicker fibers in increasing the ultimate flexural strength. Above the aspect ratio of 70, thicker fibers are more effective. This "transition" has been suggested as being attributable to the effect of interfiber interaction.[457]

The flexural strength of steel fiber reinforced concretes also increases with volume of fibers[304,418,505,521,534,567,569] to a point where the mixture contains so many fibers that they begin to interfere with each other and the mixture becomes unworkable. Experience has shown that most mixtures cannot accomodate more than 2 to 3 percent steel fibers by volume of mortar fraction of the mixture. There is also a minimum steel fiber volume which is necessary to cause the ultimate flexural strength to increase over that of an unreinforced matrix. This has been reported at between 0.2 and 0.3 percent[242] and 0.35 percent by volume.[569] Below this volume, the failure of specimens occurs simultaneously with the onset of cracking.

Swamy and Mangat[568] concluded that the maximum flexural strength for a given size, type, and amount of steel fiber is obtained in a mortar mixture. With the addition of coarse aggregate to the composite, the maximum flexural strength of the composite decreases as shown in Figure 9.4. They also showed that a nonlinear relation exists between the ultimate flexural strength and the volume of fibers for a constant aspect ratio ($\ell/d = 100$) when the aggregate contents vary. This was attributed to fiber-aggregate interaction. Snyder and Lankard[534] also concluded that the addition of coarse aggregate to a fiber-reinforced mortar reduced its first crack and ultimate strengths.

The strains at the maximum flexural load are increased substantially as a result of increasing either the length or volume of fibers.[519] Flexural strains in the tensile zone have been reported as varying from 1500 to 1900 microstrains as compared to 150 microstrains for plain concrete.

9.6.2 Effect of Glass Fibers. Flexural strength information on glass fiber-reinforced concretes is scarce, but the flexural characteristics of glass fiber-reinforced cements have been well documented. Glass fibers have been observed to increase the flexural strength of portland cement pastes,[15,66,125,198,326,332,333,334] gypsum plasters,[13,33,326,334] and concretes[343,344,577] with maximum improvements being as much as 4.9 times[344] over that of a nonreinforced concrete. The flexural behavior is similar to that of steel fiber reinforced composites with the most significant benefits occurring after first cracking.[519] First crack strength increases by a factor of 3 have also been reported for glass fiber-reinforced concrete.[344] In general, the flexural strength of glass fiber-reinforced cements, mortars, and concretes increase with increasing fiber content.[15,66,125,344,519,577] Takagi[577] using E-glass fibers and high alumina cement, found that the flexural strength of mortars increased with increasing glass fiber contents until a fiber content of 0.75 percent by weight of the matrix was reached at which time strengths began to decrease from the maximum observed. This maximum was 1.25 times greater than the flexural strength of the no-fiber mortar. The same behavior occurred in concrete but at a fiber content of 1.0 percent by weight of the matrix with the maximum strength being 1.37 times greater than the no-fiber strength. Marsh and Clarke,[344] using AR-glass in concrete found that the rate of strength increase decreased as the fiber content increased (Figure 9.5). They also experienced a point between 2.0 and 2.5 percent volume of glass fibers (based on total mix volume) where the strength improvement peaked and began to regress. In glass fiber-reinforced cements, Ali, et al.[15] showed that in thin boards produced by the spray-suction technique, the maximum amount of 1-3/16-inch long fibers that could be incorporated in a mixture was 10 percent by volume of the mixture although the maximum flexural strength improvement of 4 to 5 times the no-fiber mixture strength in general occurred at a fiber content of 6 percent. Figure 9.6 shows this phenomena as well as the effect of fiber length and storage condition of specimens on modulus of rupture.[15]

The data shown in Figure 9.6 suggest a definite effect of fiber length on modulus of rupture of glass fiber reinforced cement. Shah and Naaman[519] could find no discernible trend between glass fiber length and flexural strength in mortars. Marsh and Clarke[344] found that in concrete, longer fibers generally produced better results but adversely affected workability. Their data scatter was bad but was significantly better for ultimate strength relations than for first-crack strength relations.

Majumdar[332] noted that in cement pastes, the modulus of rupture of glass fiber reinforced cement composites decreases with age for both E and AR glass, although the AR glass performed much better than the E glass. The magnitude of the decrease in strength is dependent of the conditions of storage of the composite as can be seen from comparing the two sets of curves in Figure 9.6. It has been generally considered[162,332,333] that this reduction of flexural strength is due to alkaline attack on the glass fibers as a consequence of the high pH developed in the internal pores of the hydrated cement paste matrix. Cohen and Diamond[106] found, however, that AR glass fiber strengths remain essentially unchanged from the strength level attained after one day in a cement composite, although there was an irregular progressive decrease in flexural strengh of the composites made with these fibers beginning after one month and lasting about five to six months (Figure 9.7). They concluded that the decrease in strength should not be attributed to alkaline attack on the fibers solely but offered no explaining mechanism for the decrease.

9.6.3 Effect of Polymeric Fibers. In general, the bond of a cement matrix to a polymeric fiber is not good. Present theories[458,517,560] suggest that this low bond strength together with the low percentages of fibers usually prevalent with polymeric fibers should not improve the strength of a cement matrix reinforced with these low modulus, high elongation fibers. Because of the low volume fractions of polymeric fibers, in a matrix the total load carried by the fibers is very small. After cracking, when the strains at or near the cracks are very large, the stress in the fiber is governed by bond and the bond, being rather poor, often leads to the post-cracking strength remaining fairly constant with strain as the fibers debond and pull out.[146] Polypropylene fibrillated fibers, however, have been reported in most cases as producing substantial improvements in flexural strength. This may be due to the ability of the mortar to work itself into the twists of fiber, thus improving the bond. Figure 9.8[146] shows the relationships between flexural strength ratio and fiber volume cracking for concretes and mortars containing 2-3/4-in. fibrillated polypropylene fibers. It was noted that by increasing the matrix modulus (by increasing the aggregate/cement ratio), the increase in strength becomes less marked. Ritchie and Al-Kayyali[475] showed a slight increase in the modulus of rupture of polypropylene fiber concrete when foamed slag aggregate was used but in a similar mixture containing sintered pulverized fuel ash aggregate, there was a slight but progressive decrease in the modulus of rupture with increasing volume of polypropylene. Dardare[117] found that increases in flexural strength were dependent on both the length and volume fraction of polypropylene fibers (Figure 9.9). The maximum improvement occurred at a volume fraction of 0.6 percent with the longer fibers giving better results. When the volume fraction exceeded 0.6 percent, strengths decreased from the maximum and when more than 1.0 percent fibers were included, strengths were less than the no-fiber mixture. Nylon fibers have been reported as both increasing[630] and decreasing[367] the flexural strength. Monfore[367] reported progressive decreases in strength with nylon fiber volume for mortar reinforced with 1/4-in. long fibers (Figure 9.10). Kubota and Sakane[292] reported small flexural strength increases of mortar with vinylidene-vinyl chloride coplymer (Saran) and polypropylene fiber contents to 0.15 percent.

9.6.4 Effect of Carbon, Asbestos and Other Fibers. Waller,[615] using aligned carbon fibers in a cement matrix, was able to obtain a generally linear increase in flexural strength to a sixfold increase with increasing fiber volumes to 13 percent. When using random chopped carbon fibers in spray-suction applications, moduli of rupture as high as 4000 psi were obtained.[615] Komlous[286] found a 10 to 20 percent increase in flexural strength of concrete reinforced with 1.0 percent basalt fibers.

Asbestos fibers also increase the flexural strength of cement matrices as shown in Figure 9.11. Asbestos fibers, between 2 and 16 percent by weight of the cement, have increased the ultimate flexural strength by 1.6 times over that of the no-fiber paste.[287] Allen[21] suggested that the ultimate flexural strength of asbestos-reinforced cement pastes is influenced by both fiber content and porosity of the matrix. Dave* obtained somewhat linear increases in the flexural strength of asbestos-reinforced concrete up to approximately 9 percent asbestos by weight of the cement (Figure 9.12). Maximum strengths of 1.5 times the matrix strength were obtained with 10 percent asbestos fibers by weight of cement. With further strength additions, strengths began to decrease from the maximum. Malhotra and Winer[337] found on the other hand that additions of asbestos to the freshly mixed concrete caused reductions in the flexural strength.

9.7 TENSILE STRENGTH

No standard shape specimen of reasonable size has been used to determine the direct tensile strength of fiber reinforced cement and concrete, hence a wide disparity in results for similar materials has been published. Regardless of the test specimen, however, a distinction in behavior of the fiber reinforced concrete under tensile loading into pre-cracking and post-cracking states similar to those of flexural behavior must be made. These are separated by the first structural crack which then results in a significant change in material response. Naaman, et al.,[377] produced an idealized average stress versus elongation curve (Figure 9.13) which shows the pre-cracking state as being

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* Reference 242, pp 42-44, Discussion by N. J. Dave.

essentially elastic and after cracking being in a pseudoplastic state. Theoretically the post-cracking strength may be lower or higher than the cracking strength depending on the length and volume of fibers in the mixture.[210,377] In general, it is higher for longer or continuous fibers[377] and higher when the volume fraction of fibers is greater than a minimum volume fraction of fibers which in turn is related to the strength of the matrix, the strength of the fiber, and the length of and stress distribution across the fibers.[418]

9.7.1 Effect of Steel Fibers. A number of models and theories using the composite materials approach have been proposed for predicting direct tensile strength of fiber reinforced concrete.[33,*132,338,375,537] These models conform more realistically to observed performance than does the fiber spacing concept.[134] Figure 9.14 shows the effect of spacing of reinforcement on the cracking strength of concrete. The experimental results of Shah and Rangan,[520] Johnston and Coleman,[263] and Edgington, et al.[134] show only marginal increases in tensile strength with decreasing fiber spacing for a constant volume of steel fiber compared with the large increases predicted theoretically.[485] Cracking strain data are scarce, but in general the strains at first cracking are not significantly improved if at all over those of a no-fiber mixture. Shah and Naaman[519] produced typical stress-strain curves in tension (Figure 9.15) that showed ultimate strains of approximately 1000 microstrains. The specimens used in these tests had essentially two-dimensional fiber distribution as 1-in. fibers were used in 0.5-in. thick specimens. Pakotiprapha, et al.[418] showed only 100 microstrains with no significant post-cracking state for prisms which used 1.6-in. fibers in a 1-in. thick specimen. Johnston and Coleman[263] indicated that strain increases in mortar of up to 30 percent over no-fiber mortar are all that can be reasonably expected. Their data showed direct tensile strains in the range of 100 to 200 microstrains.

The tensile strength of steel fiber reinforced mortar increases with with increasing fiber content[134,263,377,522,574] as do the tensile

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* Discussion by J. Aveston, pp 7-9.

strains.[263] Increasing the aspect ratio also improves both these properties.[263,377] Johnston and Coleman[263] found that the tensile strength was independent of fiber cross-section. Shah and Rangan[522] noted that the length of fibers had little influence on the initial cracking strength. They also found that when a random distribution of fibers occurred, the maximum tensile loads were only 90 percent of those when the fibers were aligned parallel to the direction of stress.

9.7.2 Effect of Glass Fibers. The majority of information available on the direct tensile strength of cementitious composites reinforced with glass fibers has been developed for a simple glass fiber reinforced cement. Ali, et al.[15] and Biryukovich, et al.[66] found that the presence of fibers in glass fiber reinforced cement delayed the onset of matrix cracking, with increasing fiber contents thus producing increasing improvements in first crack strength. The amount of this improvement varied depending on whether the specimens were stored in air or water. Water storage or curing produced the best results for glass fiber reinforced mortars. Shah and Naaman[519] found similar results for glass fiber reinforced mortars.

The post cracking behavior of fiber reinforced cementitious composites is also improved with the addition of glass fibers. Figure 9.16 shows the tensile stress-strain curves of glass fiber reinforced cement composites containing 1-3/16-in. long fibers at different fiber volumes and stored in both air and water.[15] Figure 9.15a is a typical stress-strain curve for a glass fiber reinforced mortar. Figure 9.17 shows the effect of fiber content and fiber length on the ultimate tensile strength of glass fiber reinforced cement.[15] Increasing the fiber content increases the tensile strength until a critical volume of fibers is reached. This was also observed for mortar and concrete.[577] With increasing amounts of fibers, the density of the composite is reduced leading to a reduction in matrix properties. Shah and Naaman[519] found a similar phenomena in glass fiber reinforced mortar and were not able to develop a discernible trend in tensile strength with respect to the volume of fibers or the length of fibers. They concluded that the problem of reduced composite density together with increased workability caused by using longer fibers or increased volumes of fibers caused detrimental results. Results of Takagi[577] for both mortar and concrete indicate similar trends. Takagi[577] also could not find any specific tensile property trends with respect to fiber length. Post cracking strains to 1000 microstrains has been reported for glass fiber reinforced cement[15] and to 800 microstrains for mortar.[519]

9.7.3 Effect of Polymeric Fibers. Kubota and Sakane,[292] using polypropylene and vinylidene-vinyl chloride copolymer (Saran) fibers found a 40 and 20 percent maximum increase, respectively, in the tensile strength of mortars when the fiber content was 0.05 percent by weight of the cement in the mixture. With further fiber additions, the tensile strength began to continuously decrease from these maximums to values equal to or less than the no-fiber mortar strength. Brockenbrough and Davis[74] evaluated mortar briquets containing 1 percent by volume of 3/4-in., 15 denier nylon fiber and found that an average post-cracking tensile strength of 29 percent of the initial strength. After cracking, the maximum strength was reached when the crack width was 1/16 in. and decreased linearly as the crack width widened to 3/16 in. Mixtures of organic and inorganic fibers were evaluated by Walton and Majumdar[617] in an attempt to improve the tensile strength of fiber reinforced cement. Various combinations of glass fibers and nylon or polypropylene or carbon in an air-cured cement matrix produced 1-yr tensile strengths from 1300 to 1700 psi. Samarrai and Elvery[497] tested concrete prisms containing polypropylene fibers in tension where each prism had a steel rod in its center. The rod was pulled until cracks developed in the concrete with the load being related to crack width. The addition of polypropylene only provided a slight improvement in the amount of reinforcing bar stress which was necessary to cause a particular crack width to occur. Similar work was also done with steel fibers[497] and their addition gave a clear indication of a substantial increase in reinforcing bar stress.

9.7.4 Effect of Carbon, Abestos and Other Fibers. The tensile stress-strain behavior of continuous aligned carbon fiber reinforced cement for various fiber contents is shown in Figure 9.18.[33] Of significant note is the amount of strain which is from 10 to 40 times what might be expected for an unreinforced cement paste. Waller[615] showed that an increasing linear relationship between ultimate tensile strength and increasing volume of fibers exists for a cement composite containing aligned carbon fibers. He achieved tensile strengths of 14,000 to 15,000 psi with 9 to 10 percent carbon fibers.

The effect of asbestos fiber additions to concrete is shown in Figure 9.19.* The fibers increased the tensile strength, but the increases were relatively low when compared to those that might be expected from the two-dimensional random distribution of fibers found in asbestos-cement sheets. The tensile strength depended upon the fiber content and density of the specimen. The asbestos fibers in these tests were relatively short and the specimens did not exhibit any post-cracking strength.

9.8 COMPRESSIVE STRENGTH

In order to understand the manner in which fibers influence the compressive behavior of concrete, the compressive failure mechanism in concrete must be understood and unfortunately this is a very complex phenomenon. A number of different models have been proposed to describe this behavior and include lattice models,[662,663,664] interfacial behavior models,[665-668] and fracture mechanics models.[669-676] The models which appear to be best suited for understanding the role of fiber reinforcement are related to a fracture mechanics approach.

Glucklich[669] has suggested that the primary failure mode in compression is due to progressive cracking fracture and that it is the inherent heterogeneity of the concrete which is responsible for this mechanism. When a concrete specimen is subjected to a gradually increasing homogeneous compressive stress, some critical flaw or crack within the concrete will begin to grow. This growth will be spontaneous but it will soon be checked. A growing crack, approaching a particle of aggregate whose strength is greater than that of the matrix through

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* Reference 242, Discussion by N. J. Dave, pp 42-44.

which the crack is growing, must then detour around the particle. The pathway taken by the crack is usually along the interface between the aggregate and the matrix. Additional energy is required for the longer path and to overcome the bond at the interface.[635] Such a crack is now stable and cannot be a source of fracture unless the load is substantially increased. For a gradual increase of load, however, another flaw or crack, next in the order of weakness, then grows. This growth is similarly checked and the process wanders off to still another flaw or crack. In this manner, a process of progressive cracking takes place within the specimen with the load continuously increasing. Eventually enough cracks will join together to form a crack of critical size which will result in a brittle failure.

The inclusion of fibers in the concrete should greatly help the stress transfer across the flaws thus increasing the energy demand at the fiber and inhibiting the growth of the flaw. This crack arrest behavior has been suggested as the reason for increases in flexural strength and ductility for steel fiber reinforced concrete[51,485,488] and is probably applicable to all fibers whose strength and elasticity are substantially greater than that of the matrix. The maximum compressive load that can be carried by fiber reinforced concrete should be little affected by the fibers themselves except through the mechanism of crack arrest which will allow the concrete to continue to carry loads that are closer to the maximum strength of the concrete. This is somewhat speculative as the exact mechanism of fiber-concrete interaction is not known. Other factors such as fiber aspect ratio, fiber modulus, aggregate size and volume fraction, and fiber bond strength may also influence the compressive behavior.

9.8.1 Effect of Steel Fibers. In general, the addition of steel fibers to mortars results in reduced compressive strength[418,555,567,635] while additions to concrete result in slight increases in compressive strength.[62,134,135,555,567,568,635] Little or no effect has also been reported for both mortars and concretes. Regardless of whether the strengths increased or decreased, the ductility of the material is greatly improved with the fiber additions. Pakotiprapha, et al.[418]

postulated that at the ultimate condition of mortars in compression, the fibers tend to buckle thus causing premature spalling of the mortar. When this occurs, the ultimate strength of the composite should be less than that of the mortar. They showed this both analytically and experimentally with the compressive strength of steel fiber reinforced mortar being only 95 percent of that of unreinforced mortar. If fiber buckling is the assumed mode of compressive failure, lengthening fibers for a given diameter, that is increasing the aspect ratio, should result in lower strengths in mortars. It has been shown[567] that the shorter lengths of steel fiber in concrete produce greater compressive strengths. Williamson[635] has suggested that a fiber/no fiber strength ratio of less than 1.0 can be expected even in concretes when the mortar contents are in excess of 93 percent. The addition of steel fibers to shotcrete (which is a mortar) also reduces the compressive strength.[443]

The compressive strength of steel fiber reinforced concrete generally increases with fiber volume. Maximum increases have varied between 10 and 25 percent. Edgington, et al.[134] have suggested that these small increases in strength could have been more cheaply achieved by decreasing the water-cement ratio providing that the increased ductility of the concrete due to the fiber addition is not also a consideration. No unique strength increase pattern exists for all aggregate contents with the variation of compressive strength being different for each aggregate volume.[568] Swamy and Mangat[567] also found that no unique relationship exists between compressive strength and fiber aspect ratio. They did relate the maximum compressive strength to an optimum fiber spacing value of 0.177 in. Williamson[635] pursued a similar approach and found the fibers to be ineffective at a spacing of 0.40 in. or greater. In general, the maximum compressive strength does not occur at the same fiber spacing that the maximum flexural strength occurs.[568]

The effects of age on the compressive strength of steel fiber reinforced concrete with regard to the contribution of the fiber can be considered to be insignificant.[568] Williamson[635] in evaluating 3- by 6-in., 4- by 8-in., and 6- by 12-in. cylinders concluded that the

Weibull effect of specimen size could be ignored as there was no significant difference in the compressive strength of the three sizes of cylinders. Steel fiber reinforced concrete is strain rate sensitive in compression with lower strengths being obtained for slower deformation rates.[434] Limited data[249] indicate that strengths of steel fiber reinforced concrete cores were approximately 14 percent less than cast cylinders. The effect of the direction of external vibration on the compressive strength of steel fiber reinforced concrete is insignificant but it does greatly influence the post-cracking behavior as indicated in Figure 9.20.[133]

9.8.2 Effect of Glass Fibers. With glass fibers, the compressive strength of concrete increases as the volume percentage is increased until a maximum is reached. Then further fiber additions result in strength decreases[344,577] as shown in Figure 9.21. This occurs for both E and AR glass fibers. Takagi[577] found that E glass fibers in high alimina cement mortar linearly increased the compressive strength with increasing fiber content when the fibers were 1/8 in. long. For greater fiber lengths, the increase was linear to approximately 0.4 to 0.6 percent fibers at which time a strength reduction occurred. No reductions fell below the original no-fiber strength, however. Marsh and Clarke[344] found that for AR glass fiber lengths of 1/2 to 2 in., there was no notable difference in the maximum compressive strength obtained although the percentage level at which it occurred varied between fiber lengths.

9.8.3 Effect of Polymeric Fibers. The addition of polypropylene fibers to concrete generally does not affect the compressive strength[9,94,110,292] but they do result in a controlled compressive failure (Figure 9.22) after the matrix has cracked.[74,449] Nylon fibers have been reported as reducing compressive strength with increasing fiber content (Figure 9.23)[74] although nylon fiber reinforced concrete with compressive strengths greater than 12,000 psi have been produced.[479] No data on carbon fiber reinforced concrete was found as this fiber is usually used as a unidirectional fiber for improved bending and hence compressive strength is generally not considered. The same was true for asbestos fiber reinforced composites.

9.9 DIMENSIONAL CHANGES

Hydraulic cements such as portland cement will expand if allowed to set and then harden in water. On drying they undergo a shrinkage which is influenced by many factors such as aggregate content, elastic properties of the aggregate, water content of the mixture, and others. This shrinkage is, in part, reversible if the cement paste is subsequently rewetted. In general, very little consideration has been given to the effect and influence of fiber additions on the shrinkage or dimensional stability of fiber reinforced cements, mortars, and concretes. This important consideration will influence many future applications and additional work is needed. The limited data available for all types of fibers has suggested that the contribution of fibers in reducing drying shrinkage is small and may be negated when mixtures are adjusted to higher water and cement contents to accomodate higher fiber loadings.

Edgington, et al.[134] found that the shrinkage of concrete over a period of 3 months on specimens subjected to various curing environments was unaffected by the presence of steel fibers. They also found that the addition of steel fibers in concrete did not reduce the compressive creep strains of the composite over a 12 month period. In both cases, it was concluded that because the additions of steel fibers were small (2 percent by volume) compared to the aggregate volume (70 percent by volume), the aggregate was the controlling factor in the behavior. McKee[355] also concluded that steel fibers had no significant effect on the creep characteristics of portland cement. Elvery and Samari[137] found that shrinkage cracks in no-fiber control slabs occurred after 21 days of drying while slabs containing steel fibers experienced no cracks after 520 days of drying. Swamy[552] when reporting that the presence of fibers in concrete caused a distinct reduction of 10 percent in shrinkage also noted that while shrinkage was reduced, what was more important was the ability of the fibers to reduce shrinkage cracking, as was similarly observed by Elvery and Samari.[137]

Grimer and Ali[198] reported that although the drying shrinkage of glass fiber reinforced cement was significantly reduced with increasing glass content, the amount of reduction at a fiber content of 10 percent by weight was only 20 percent. This suggested that the fibers modified the matrix very little. Dardare[117] observed an increase in shrinkage of polypropylene fiber reinforced concrete when high amounts of polypropylene fibers were used. This was attributed to the increased water contents necessary to promote adequate mixing. Briggs, et al.[73] observed tenfold reductions in both expansion and shrinkage of cements when 5.6 percent by volume of high modulus graphite fibers were incorporated in it. The shrinkage of the matrix in or at 60 percent RH was reduced by the same margin by which expansion in water storage at 68°F was reduced.

Swamy[552] reported that in tension, steel, glass, and polypropylene were all effective in reducing creep to various degrees although, as noted above, steel was not effective for creep in compression. Carbon fibers were found to reduce flexual creep deflection by a factor of 6 at 2 percent volume of fibers.[73] At 9 percent volume of fibers, the creep resistance was increased 40 times. The creep recovery of the matrix was also found to be aided by the fibers.

9.10 DURABILITY

Durability is the degree of retention of the initial mechanical properties of the hardened fiber reinforced concrete with time. For a material to require zero maintenance the durability is a prime consideration. The development of accelerated testing procedures for concrete has had a controversial history and it is generally agreed that it is best to await the results of long-term testing. Information on the long-term durability of fiber-reinforced concrete is scarce, however, although several studies are in progress.

The durability of fiber reinforced concrete is related to both the properties of the concrete constituents and the interactions between them. The hardened cement paste can be considered a continuous constituent while the fiber and aggregate are dispersed or discontinuous constituents. The cement matrix normally contains pores, voids, and microcracks which result in a poor bond to the fibers and can allow the intrusion of aggressive liquids into the concrete. It also has a low value of tensile strain at failure which results in a heavily cracked matrix when the fibers are first beginning to develop their full load carrying capacity. To improve these situations, the matrix should be formulated to be dense and impermeable, to have a low absorption and shrinkage, and have an adequate pore distribution. This can be achieved in part by using the lowest water-cement ratio possible and proper consolidation. The physical properties of the hardened cement also change with time, and this can negatively affect some of the properties of the composite such as impact resistance. The cement gel in the matrix is hydroscopic in nature, and its water content increases and decreases according to the relative humidity of the surrounding atmosphere. These moisture changes produce volume changes of the composite and keep the interfacial region between the matrix and fiber in an active state for a long time.

The normal hydration of the portland cement results in the liberation of calcium hydroxide which quickly forms a saturated solution in water of very high pH (12-14) which then collects in the pore space of the cement gel. The very high pH solution which results from the presence of alkali hydroxides tends to protect steel but has a degrading effect on glass. The hardened cement itself is basic in nature and will be attacked and decomposed by acids although the rates of attack may be very slow for weak or dilute acids.

9.10.1 Effect of Steel Fibers. Intuitively, steel fibers would be expected to rust when in concrete. Whether they do or not depends on the condition of the concrete. Uncracked steel fiber reinforced concrete has shown little evidence of deterioration when exposed to a variety of exposure conditions.[213,527,*] Cracked steel fiber concrete on the other hand can be expected to lose its effectiveness with prolonged exposure.[53,213]

Shroff[527] found insignificant corrosion by salt water on portland

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* Horeczko, G., "Potential Use of Fibrous Concrete in the Port Facilities of Los Angeles Harbor," unpublished, Los Angeles Harbor Department, Wilmington, California, May 1976.

cement mortar reinforced with 2 percent steel fibers and no change in flexural strength after 90 days rotation in and out of saturated salt water solution. Horeczko (see note) made both beams and cylinders of steel fiber reinforced concrete where his variables were cement contents of 658 and 846 lbs per cubic yard and fiber contents of 100 and 200 lbs per cubic yard. After 28 days fog curing the specimens were set in the ocean for observation at 2, 10, and 20 years age. The specimens evaluated at 2 years age all had a mottled and discolored surface due to oxidation (rusting). Sliced sections revealed no internal rusting below the mottled surface. Compressive and flexural strength ratios to nonexposed controls were as follows:

	2 year exposure results			
Cement Content, lb/cu yd	658		846	
Fiber Content, lb/cu ys	100	200	100	200
Compressive strength ratio	1.01	1.03	1.08	1.16
Fluxual strength ratio	1.08	1.45	1.22	1.48

The control strengths for the 658 and 846 lb/cu yd cement contents mixtures were 6420 and 8710 psi compressive and 850 and 1010 psi flexural, respectively. Hannant and Edgington[213] studied uncracked cylinders made from a good concrete for marine exposure and a permeable lightweight aggregate concrete. Both contained 1.22 percent by volume of steel fibers. These were exposed after 7 days moist curing to three different environments: (1) mild temperature, dry, and mild air pollution; (2) cold temperature, exposed, coastal; and (3) sewage outfall, very high pollution with sulphur compounds. The cylinders were studied at four ages to 57 months to determine the depth to which corrosion occurs with time. The results indicated that normal weight concrete made for marine exposure, when uncracked, provides adequate protection to the internal fibers for at least five years. The degree of surface rusting was more severe for exposures 2 and 3 and was more pronounced for the lightweight concrete.

Hannant and Edgington[213] also looked at cracked beams exposed in the same conditions as their cylinders. These beams used two different types of steel fibers with one having twice the surface area to volume ratio as the other. The volume percentage fibers in each case was 1.4 percent. The beams were wet cured 7 days and then cracked on the eighth. The average crack widths varied from 0.003 to 0.02 in. with the beams being cracked to an average 80 percent of the depth of the beam. Attempts were made to measure the rate of carbonation into the crack and determine the capacity of beams to resist load in the corroded state. It was concluded that the cracked concrete experienced a greatly increased rate of carbonation and of fiber corrosion local to the crack when compared to uncracked beams. After 11 months exposure, the beams appeared to be getting stronger. This was attributed to improved fiber-cement bond and autogenous healing of the crack. The authors speculated that when the fiber strength is eventually reduced by rusting to below the fiber pull-out load for the majority of the fibers, the failure mechanism would change and the beams would fail in a more brittle manner. They concluded that it would be unwise to extrapolate to a critical exposure duration. Batson and Obszarski[53] evaluated the strength reduction of steel fiber reinforced concrete exposed to flowing salt water environment under laboratory conditions in order to simulate conditions of salt for ice control. They sawed a notch to a depth of 40 percent of the specimen and propagated a sharp crack from the notch tip. The notch was held open by wedges to create a high stress concentration at the tip of the crack. The notch and crack were continuously flushed with salt water for the duration of test. The concrete was made with fiber contents of 1, 1.5, and 2 percent by volume. The authors related the performance of the beams to a critical strain energy release rate using fracture mechanics concepts and concluded that this rate decreased at a decreasing rate for exposures up to 176 days. At 176 days the critical strain energy release rates for concretes containing 2, 1.5, and 1.0 percent were 89, 81, and 66 percent, respectively, of the nonexposed controls. In general, all specimens observed at ages of 113, 148, and 176 days showed visible evidence of corrosive action. Any exposed fibers had rusted.

9.10.2 Effect of Glass Fibers. Borosilicate E glass fibers have a known history of deterioration in the alkaline environment of a cement paste.[66,198] To alleviate this problem, new alkali resistant (AR) glass fibers containing zirconia (ZrO_2) have been produced.[333,336,344,592] While vastly better than E glass, these "alkali resistant" fibers are not totally immune from strength degradation when placed in a cementitious matrix. The exact causes of this degradation and the degree of severity for each cause are a matter of some speculation. Swamy[552] noted that the durability of glass fiber reinforced cement (GRC) composites was affected by both alkali attack and surface flaws or damage produced either during fabrication or from crystal growth on the fibers. Majumdar[331] in studying single filaments of AR glass in cement extracts, found a tensile strength reduction of approximately 40 percent occurring within 6 months. There was little change in the next 1.5 years. During the 2-year period, the modulus of elasticity remained relatively unchanged. Cohen and Diamond[106] published some Owens Corning Fiberglas Corporation data that indicated that the strength of AR glass strands (not single filaments) was degraded over a period of a few weeks but then became stable against further strength loss (Figure 9.24). They also looked at the tensile strength of fibers taken from cement composites. These fibers experienced a loss from 30 to 45 percent of their original strength after mixing and one day curing. No further losses occurred even after 500 days exposure. This finding conflicted with observed reductions in flexural strength which have been observed to occur in glass fiber reinforced cement composites from several weeks to months after fabrication. The fact that the fibers themselves were not losing strength after one day led Cohen and Diamond to conclude that alkali attack is not primarily responsible for the intermediate term flexural strength reduction of the glass fiber reinforced cement. They did suggest, however, that because of the very high pH levels attained in the cement matrix, alkali resistance is still very important to the potential long-term durability.

9.10.3 Effect of Polymeric and Other Fibers. Organic fibers such as polypropylene, nylon, and polyethylene are generally resistant to acids and alkalies, and most of them are relatively unaffected by water. Fibers such as cotton, rayon, acrylic, and polyester are subject to alkaline attack and are not very effective. Carbon fibers appear to perform well in a cement environment. Ali, et al.[14] studied carbon fiber reinforced cement prisms in flexure for curing conditions under water at 64°F, in air of 40 percent RH at 64°F, and in water at 122°F. They found no change in the modulus of rupture up to one year exposure for the water and air storage at 64°F. Water storage at 122°F resulted in a slight but continuing decrease in strength up to one year exposure. Briggs, et al.[73] also found good durability performance with carbon fiber reinforcement. Table 9.3 is a summary of the environmental durability of the more common fiber types.[162]

9.10.4 Freeze-Thaw Resistance. The additions of fibers to concrete cannot be expected to result in an improved freeze-thaw resistance over what the concrete was prior to the fiber additions. This is to say if the basic concrete was not frost resistant, the fiber reinforced concrete will also not be frost resistant even though the fibers may cause the concrete to hang together a little longer after the freeze-thaw action begins to break it up. Table 9.4 contains freeze-thaw data developed at the U. S. Army Engineer Waterways Experiment Station which supports this view. The concretes which were durable had an average entrained air content of 7.6 percent. Those that were not had an average air content (mostly entrapped) of 2.4 percent. Kaden[266] found that for field placed steel fiber reinforced concrete having air contents of 5.9 and 3.3 percent the DFE at 300 cycles was 73 and 85, respectively. The concrete with the 3.3 percent air content had a water-cement ratio of 0.35 and was not evaluated until 44 days old, at which time its compressive strength was approximately 8000 psi. Briggs, et al.[73] found good freeze and thaw resistance with carbon fiber reinforcement of cements.

9.11 ABRASION RESISTANCE

Information on the abrasion resistance of fiber reinforced mortars and concrete is scarce. Tests simulated by sand blasting and electrically driven rotary steel brushes showed that steel fiber reinforced concrete exhibited less spread and depth deterioration with increasing fiber content.[364] Tests made with a National Bureau of Standards wear testing machine indicated that steel fiber reinforced concrete slabs with pea gravel and 2.5 percent by volume fibers abraded to a depth 27 percent less than plain concrete with the gravel.[5]

In a related area of performance, Mikkelmeni[364] found that steel fiber reinforced concrete surface possessed up to 15 percent higher rolling resistance than plain concrete under dry, wet, and frozen surface conditions. The coefficient of static friction appears to be independent of the steel fiber content.[364]

9.12 TOUGHNESS

The toughness of concrete is related to crack growth and is greater than the toughness of cement paste alone because of the more extensive microcrack growth in concrete due to the presence of aggregate (See Section 9.8). When fibers are present in the concrete, they also interfere with crack growth and a considerable amount of extra energy is necessary to stretch and debond the fiber in order to allow the crack to grow. This requirement for additional energy to cause cracks to grow and thus produce failure is described as improved toughness of the material.

The toughness of fiber-reinforced cements and concretes can be described quantitatively as the area under the compression or tension stress-strain curve, or the area under the load-deflection curve. To effectively compare the relative toughness of various fiber reinforced composites, the definition of toughness must be more definitive however and should be related to prescribed values of either stress, strain, or deflections. This generally has not been done although Johnson[261] suggested defining toughness, based on load-deflection curves, as the area under the curve to the maximum flexural stress attained, or up to a specified deflection consistent with the cracking allowable in service. Another technique suggested for defining toughness* has been to report the area under the stress-strain curve from the start of the test to the point of twice the strain which occurred at ultimate load. Henager** proposed an energy absorption index whereby a fiber reinforced beam (ASTM C78, 4 in. by 4 in. beam using third-point loading) is deflected for 0.075 inches for the 12 inch span and the area under its load deflection curve measured. This area, expressed in inch-pounds, would be divided by the energy area observed for a no-fiber concrete specimen of the same comparable mixture tested to failure. The resulting parameter would be called an energy absorption index.

There is not much quantitative data available on the toughness of either plain concrete or fiber reinforced concrete but good indicators of toughness improvements, although not specifically noted, are available each time a load-deflection curve or stress-strain curve for fiber reinforced concrete is published. Figure 9.25 is indicative of this type of information.[555] Represented are the load-deflection curves of steel fiber reinforced mortar containing no fibers and volume percentages of 1, 2, and 3, percent fibers. As can be seen, the increasing fiber contents produce increases in the area under the load-deflection curve and hence an increase in toughness of the material. Similar information is available in Figure 9.16 for glass fiber reinforced cement. It is reasonable to assume that some of the same parameters that influence the ultimate strength of fiber reinforced concrete will also influence its toughness. These include volume percentage of fibers, aspect ratio, and fiber orientation. Figure 9.26 shows the influence of volume of fibers on toughness in flexure.[521] The increase in toughness was as much as twenty times for 1.25 percent fibers by volume while for the same volume of fibers, the increase in strength was less than two times.

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* Private communication from E. K. Schrader, Walla Walla District, U. S. Corps of Engineers, Walla Walla, Washington, 15 March 1976.

** Private Communication from C. H. Henager, Battelle Northwest Development Laboratories, Richland, Washington, 23 March 1976.

Shah and Rangan[521] found that up to a point ($\ell/d = 75$), increasing the length of fibers continuously increased the toughness of the concrete. This was true for both flexure and direct tension test results. They also found that in tension, increasing the amount of fibers aligned in the direction of loading also increased the toughness as shown in Figure 9.27. The orientation effect on the stress-strain curve, and hence toughness, as caused by external vibration of concrete is shown in Figure 9.20. Regardless of the factors influencing toughness and the methods of determining it, significant improvements in the toughness of glass fiber-reinforced cements,[16,19,20,23,326,333,334,604] glass fiber-reinforced mortars and concretes[160,343,577] glass fiber-reinforced gypsum[13,326,334] and steel fiber-reinforced mortars and concrete[69,160,263,484,485,487,521,522,569] have been noted. Although not specifically described as toughness, the stress-strain behavior of polymeric fiber reinforced mortars indicates that their toughness is also significantly improved. It is generally believed that the toughness of fiber reinforced concrete is at least an order of magnitude higher than that of plain concrete.[62,261,408,521,522,632] Toughness, when described as fracture toughness, has also been related to crack growth through the linear elastic approach of Griffith.[677] Using this approach Romauldi, et al.[481,489] determined analytically that the fracture toughness of steel fiber reinforced concrete should be an order of magnitude greater than that for unreinforced concrete. Parimi and Rao,[423] using an energy approach, developed an expression for fracture toughness which indicated dependence on the modulus of the fiber, the volume of fibers in the mixture, the bond strength of the matrix with the fiber, the fiber length, and the fiber diameter. They believed that the relationship of Romauldi, et al.[481,489] greatly overestimated the fracture toughness of a fiber reinforced composite. Data to support this is scarce however. Brown[78] found that in examining the fracture toughness of glass fiber-reinforced cement composite, the toughness was similiar to that of an unreinforced cement until the start of crack growth. The toughness then increased linearly with crack growth at a rate proportional to the fiber content. When 5 percent by volume of glass fibers was used, the fracture toughness increased by a factor of 4 over the unreinforced matrix.

9.13 FATIGUE STRENGTH

Under load, practical fiber-reinforced cement composites crack at very nearly the matrix failure strain which is of the order of 0.02-0.06 percent, although their ultimate failure strength may be several times that of the matrix.[331] When such a load is sustained over a long period of time the composite undergoes static fatigue and for stresses in excess of those at the proportional limit or first crack load as shown in Figure 9.1, deterioration of the strength will take place. The same type of behavior can probably be expected to occur for fiber-reinforced cement, mortar, and concrete. Conversely brittle materials are usually free of dynamic fatigue under oscillating or reversible loads, and this is generally true of all composites.[552] The cracking at the applied stress level will however, have a distinct influence on the fatigue strength

<u>9.13.1 Effect of Steel Fibers</u>. Tests on steel fiber concrete generally show that the fatigue strength increases with fiber volume. Comparisons of results is difficult however because beam sizes, loading conditions, and failure criteria have varied among researchers. Both flexural[36,38,50,238,299,355,481] and compression fatigue[462] have been measured however. Romauldi[481] found that for non-reversal loading, the flexural fatigue endurance limit was approximately 90 percent of the static tensile strength. Post fatigue static flexural strength was 10 to 30 percent greater for similar beams with no fatigue loading history. One explanation that was offered for this behavior was that the cyclic loading reduced the initial residual tensile stresses due to shrinkage of the matrix by accelerating the creep.[481] Batson, et al.[50], working at steel fiber contents of 2-3 percent, observed that the endurance limits for complete reversal and non-reversal of loading to 2 million cycles were 74 and 83 percent of the first crack static flexural strength. The relation between first crack static flexural strength and number of loading cycles for the complete reversal loading is shown in

Figure 9.28. The significance of the difference between the 74 and 83 percent values for the two types of loading was not clear because different size fibers were used. The fatigue strength was reduced to 50 percent of the first crack strength at 10 million cycles for the non-reversal type loading. As was the case in Romauldi's work,[481] Batson, et al.[50] found that the post-fatigue static flexural strength (924 psi) was greater than the pre-fatigue static flexural strength (694 psi).

Ramey and McCabe[462] studied the compression fatigue of steel fiber reinforced concrete and concluded that there was no apparent difference in the uniaxial fatigue strength of fiber reinforced concrete and conventional concrete with the exception that the explosive failure normally associated with the conventional concrete did not occur for the fiber reinforced concrete.

9.13.2 Effect of Glass Fibers. Allen[20] has performed cyclic tension measurements on glass fiber-reinforced cement laminates. A typical cyclic stress strain curve from these tests is shown in Figure 9.29a. For clarity, the vertical scale is halved. The envelope of these curves is roughly comparable with the simple stress-strain curve for this material. The tensile strain consists of a permanent deformation and a recoverable elastic strain, with their respective magnitudes being related to the maximum total strain, ε_{np}, (Figure 9.29b) to which the specimen has been subjected. As ε_{np} increases, the permanent residual strain rises and the stiffness of the specimen decreases.

Hibbert and Grimer[228] determined the dynamic fatigue life of glass fiber reinforced cement (V_f = 4 percent) in four point bending using a frequency of 3 H_z with the stress varying between zero and a predetermined maximum. Although it was not possible to determine the endurance limits, at stresses up to 90 percent of the proportional limit stress in static bending, a fatigue life of one million cycles was recorded for specimens stored one year under various environmental conditions including natural weathering. Some of these results are shown in Figure 9.30. For a given stress, the fatigue life increased with increasing fiber content, but at levels above the proportional limit, the fatigue life was reduced appreciably with its magnitude depending on the applied stress.

Majumdar[331] noted that specimens of glass fiber-reinforced cement (V_f = 4 percent) are being weathered on an exposure site at Garston, England under a bending load to establish their static fatigue behavior. At stress levels below the proportional limit, no significant reduction in bending strength of the composite had been observed after one year although some permanent deformation had taken place.

9.13.3 Effect of Carbon Fibers. Briggs, et al.[73] examined aligned graphite fiber composites for their static fatigue behavior at levels of stress between 8 and 27 percent of the ultimate strength of the composite. The experiments were conducted in different environments (air, wet and dry, and freeze and thaw) and lasted as long as 26 weeks. All specimens (V_f = 7 percent) showed a consistent deterioration in strength while in the loaded condition and this loss ranged from 17 to 27 percent with respect to the unloaded condition. Dry air environment was found to be the most deterious irrespective of the level of stress.

Briggs, et al.[73] also reported on the effect of repeated loading on the bending strength of aligned graphite fiber cement composites. Specimens made with either of two types of fiber at fiber contents of 8 percent were subjected to both low-frequency (30 cycles per minute) and high-frequency (2000 cycles per minute) loading. The results are shown in Figure 9.31. The tendency of the curve to level off around 10^8 cycles suggested that the material had fatigue limits in the region of 10,150 and 11,600 psi for the Type I and Type II fiber composites respectively. These limits occurred at stresses much higher than the matrix cracking stress thus suggesting that the cracking of the matrix at relatively low stresses may not be harmful to the composite performing at stresses below its fatigue limit.

9.14 IMPACT RESISTANCE

The use of fibers in a cement matrix, mortar, or concrete not only improves the toughness of the composite as described in Section 9.12, but the presence of the fibers prevents the total disintegration and shattering of concrete normally associated with shock loads. With fibers, the cracks which form cannot extend without stretching or debonding or both, and because of this behavior, substantial energy inputs are necessary before complete fracture of the material can occur.

The most important property influencing the energy absorption characteristics of a composite is the interfacial bond of the fiber.[552] This is not to say that good bond is desirable for this characteristic, however. High impact strength is believed to be due to rather poor interfacial bond strength[123,124] in the composite which means that a large portion of the energy transmitted to the material during impact is expended in the progressive debonding of the fibers. The role of the interfacial bond in cement composites is analogous to that in plain concrete[571] where the energy dissipation due to friction damping at the interfaces and the inelastic deformation at the interface discontinuities contribute substantially to the fracture energy of the composite.[678] The shape of the fiber and its aspect ratio are then important parameters as they determine the amount of fiber pull-out (see Section 9.5). The orientation of the fibers is important only in its ability to control cracking. As the work of fracture is related to impact strength and is influenced by the volume of fibers, it follows that the impact strength should also be influenced by the concentration of fibers in the composite.

The resistance to impact has been measured empirically by dropping a ball on plate specimens, qualitatively by explosive detonations or by firing projectiles into specimens, or with a little more meaning by the conventional pendulum test methods (Izod or Charpy). In most cases what is measured is the energy expended to cause complete separation of the material. In contrast to strength development, high impact resistances are usually associated with matrices reinforced with fibers of low moduli[140,183,184,475,538,560,636,658] and poor bonding between the fiber and matrices.[13,333,438,490,495] High moduli fibers are also effective but the relative improvements are not quite as great.

9.14.1 Effect of Steel Fibers. Johnston[261] examined the impact resistance of steel fiber reinforced mortars using a Charpy Tester. Variables included fiber content, aspect ratio, and fiber shape. While data trends existed, Johnston[261] felt that the scatter of his data was too wide for the small number of specimens tested to permit the establishment of a precise relationship between impact resistance and his variables. He suggested that nonuniformity of fiber distribution may contribute to data scatter of this sort, particularly for small specimens. Edgington, et al.[134] also used a pendulum type impact machine to evaluate 4-in.- by 4-in.- by 20-in.-long specimens of steel fiber reinforced mortar and concrete made with different shapes and strengths of steel fibers. Concrete aggregate maximum sizes were approximately 3/8 and 3/4 in. The results are shown in Figure 9.32. It is clear that the impact resistance of steel fiber reinforced concretes tested in this manner was considerably increased. Of the four fiber types tested, a 0.020-in. diameter by 2-in.-long (aspect ratio = 160) high tensile crimped fiber proved most beneficial. Increases in impact resistance of more than 400 percent at less than 1-3/4 percent by volume were measured with this fiber.[134] Ritchie and Al-Kayyali[475] found that steel fiber increased the impact resistance of lightweight concrete by as much as three times.

With explosive loading, a shock wave travels through a wall as a compressional wave and is reflected on the opposite face as a tensile wave which then causes spalling and disintegration of concrete. Williamson[630,632,636] has shown that while conventionally heavily reinforced slabs disintegrate completely under explosive loading, the inclusion of steel fibers in the concrete reduces the fragment velocity by about 20 percent and more importantly, causes the slabs to retain their integrity without producing secondary fragments of sufficient mass and velocity which might cause further damage. Swamy[555] has reported improved impact resistance of steel fiber reinforced slabs subjected to repeated drop ball tests. Failure in these tests was defined as occurring only when a hole was punched through the slab rather than by shattering of the slab into fragments.

9.14.2 Effect of Glass Fibers. Most of the work involving impact resistance and glass fibers has been done on glass fiber reinforced cement composites. Ali, et al.[15] used an Izod tester to evaluate specimens of glass fiber reinforced cement containing various lengths and volumes of glass fibers. The results are shown in Figure 9.33. The composite strength increased with increasing fiber contents in both air and water curing up to the limit of study which was 8 percent volume of fibers. They noted that air curing produced a bigger increase in impact resistance than water curing and attributed this to both an increase in the porosity of the composite caused by the loss of moisture due to drying and the fiber additions themselves. With increased porosity, a greater proportion of the fibers thus becomes available for pullout, which in turn controls the work of fracture or impact strength of brittle matrix composites.[15] The 28-day impact strength of glass fiber reinforced cement having 6 percent volume glass fibers was 15 to 20 times that of the unreinforced matrix. Ali, et al.[15] also noted that the impact strength improved with increased fiber length because of improved "pullout" considerations. Steele[541] showed that there was a degradation in impact resistance of glass fiber reinforced cement with time regardless of whether it was cured in air or water. This loss in impact strength is shown in Figure 9.34 for both E-glass and A-glass fibers. Even with the losses, the AR-glass composites have 15 times the impact resistance of the no-fiber matrix and approximately 5 to 10 times that of asbestos reinforced cement.

Combinations of glass fibers and polymeric fibers such as polypropylene and nylon, when used to reinforce cements, also improve the impact resistance of the composite.[617] Some typical data are shown in Table 9.5 and were determined from Izod pendulum tests. The no-fiber matrix had an impact strength from 95 to 135 ft-lb/ft^2.

The impact strength of glass fiber reinforced gypsum with approximately 12 percent by weight fiber content has been reported to be between

20 to 30 times that of unreinforced gypsum[13,326,333,334] and to be up to 5 times greater than that of asbestos fiber-reinforced gypsum.[13,326,592]

9.14.3 Effect of Polymeric Fibers. Goldfein[140,184] studied the effect of many plastic fibers and found that nylon and polypropylene fibers can increase the impact resistance of concretes by up to as much as 30 times that of unreinforced concretes. Similar results were found by Williamson.[630] Walton and Majumdar[617] found similar behavior for cements reinforced with these fibers as shown in Table 9.6. Polypropylene is being widely used in the United Kingdom and Europe to improve impact resistance of cements, mortars, and concretes used in applications where this property is of prime consideration such as in precast piles. The impact resistance of polypropylene reinforced piles has been reported as being improved 40 percent in comparison with steel mesh reinforced pile shells.[140]

Ritchie and Al-Kayyali[475] found that polypropylene and steel fibers were both effective in improving the impact strength of lightweight concrete as shown in Figure 9.35. Using a modified Izod test machine the results from the testing of 4-in.- by 4-in.- by 20-in.-long specimens of fiber reinforced concrete indicated improvements with increasing volume of fibers and a dependence on aggregate type. The improvement was most significant for foamed slag aggregate where the addition of polypropylene doubled the impact resistance and steel fibers tripled it.[475]

9.14.4 Effect of Asbestos, Carbon, and Other Fibers. Asbestos fibers can double the impact resistance of cements when incorporated into the matrix at 10 to 11 percent by weight.[438] Significant improvements have also been noted for asbestos reinforced gypsum cement.[13,326,592] Only small improvements (1.2 to 2.6 times the no-fiber strength) have been noted in impact strength of carbon fiber reinforced cements.[617] Combinations of carbon fibers and other lower modulus fibers have shown substantial improvements, however.[617] Vegetable fibers such as coir and jute, although degraded by alkali in the cement, improved the impact resistance of concrete by three to four times.[538]

9.15 PERMEABILITY

No information on the permeability of fiber-reinforced concrete could be located. There is nothing in the proportioning or production of fiber reinforced concrete that suggests that the permeability should be significantly different than normal portland cement concrete. In fiber reinforced cement composites where vacuum processing or suction techniques are used to remove water of convience prior to hardening of the cement, the permeability should be improved. Whenever water-cement ratios are raised in order to improve workability of a fiber reinforced mixture, the permeability will probably be adversely affected.

USE OF FIBER REINFORCED CONCRETE

The use of fiber reinforced cements has been actively pursued in the United Kingdom and Europe for many years for principally non-structural purposes. These uses have only recently begun to spread to North America. Since the early 1960's, interest in the United States has been principally directed towards structural applications using steel fibers as the reinforcement. The most reported use of the steel fiber reinforced concrete has been in the areas of pavements*, bridge decks,[148,501,503,656] and floors[7,139,508,641]. As this report is concerned mainly with the structural use of paving materials, the uses of fiber reinforced concrete discussed in the following sections will be restricted to pavement and pavement related constructions and uses.

9.16 BRIDGE DECKS

It has been suggested that fibrous concrete could be applied to a number of different situations within bridge construction.[55] These include permanent soffit formwork as precast lightweight panels, total

*References: 25, 26, 74, 79, 108, 128, 152, 153, 156, 158, 188, 190, 192, 193,197,249,260,261,265,284,308,309,312,321,346,346,353,360,467,425, 426,428,431,441,471,472,473,539,540,588,589,606,612,613,614,631,634, 648,649, and 650.

depth construction, wearing surfaces, composite construction combined with conventional reinforcement, expansion joint nosings and curb edgings where high impact and abrasion resistance requirements occur, and shotcrete repairs to existing structures. Wearing surfaces has been the most common application.

Ten fiber-reinforced bridge deck surfacings were constructed in the United States between 1972 and 1975. In all of the projects, steel fibers were used at fiber contents varying from 0.75 to 1.5 percent by volume.

9.16.1 Winona, Minnesota The first job was a fibrous concrete overlay of a precast bridge in Winona, Minnesota in August, 1972,[501,503] Severe scaling of the surface of the 30 by 95 ft precast deck necessitated the repair. The surface scale was removed to a depth of 1-1/2 in. and a steel fiber concrete overlay placed over the deck with no joints. Thicknesses were 2.5 to 4.0 in. The fiber content was 200 lb of 0.010-in. diameter by 0.5 in. long fibers per cubic yard of concrete. Reports after one year[55] indicated that the overlay was successfully combating the effects of salting and studded tires.

9.16.2 New Cumberland, Pennsylvania Also in August, 1972, the deck of a steel truss bridge in New Cumberland, Pennsylvania was overlayed with steel fiber concrete. This deck, which was 155 ft long and 40 ft wide, had been frequently overlayed with asphaltic concrete. After removal of this overlay and any unsound concrete, a steel fiber overlay was placed to a depth of 2 to 5 inches depending on how much old concrete had been removed. A cement paste bonding agent was used to fully bond the overlay. The overlay was done in one placement with no joints. The fiber content was 200 lb per cubic yard of concrete of 0.010 in. by 0.022 in. by 1.0 in. long steel fibers. Traffic was allowed on this overlay after 7 days. After one year of service the overlay had no major defects however a few cracks had developed.[186] Most of these developed at areas where the concrete overlay varied abruptly in thickness. The cracks measured from 0.01 to 0.03 in. in width.[186]

9.16.3 Cedar Rapids, Iowa Early in October, 1972, one lane of a wood truss railroad viaduct on First Avenue in Cedar Rapids, Iowa, was replaced with an unbonded fibrous concrete overlay. The deck had its old asphaltic concrete removed down to the wood plank decking. Double polyethylene sheeting served as the bond breaker. The slab placed was 152 ft. long by 11 ft wide by 3 in. thick and contained no joints. A fiber content of 150 lb of 0.025 in. diameter by 2.5 in. long fibers per cubic yard of concrete was used. Traffic was allowed on the overlay after four days. After three years of 30,000 vehicles per day the overlay was essentially crack free.[1]

9.16.4 Pittsburgh, Pennsylvania. In July, 1973, a five-lane prestressed box beam bridge in Pittsburgh, Pennsylvania, was overlayed with 3-in. of steel fiber concrete.[308] The 40-ft by 60-ft overlay was bonded to the old deck using an epoxy bonding agent. The mixture had a fiber content of 200 lb of 0.010 in. by 0.022 in. by 1.0 in. long fibers per cubic yard of concrete.

9.16.5 New York City, New York. The Dyckman Street Bridge in New York City was overlayed in March, 1973, using 10 to 12 in. of steel fiber concrete which was bonded to the old deck.[308] The overlay was four lanes wide and 250 ft long and contained 4 in. by 12 in. steel mesh at the midpoint of the overlay section. Steel fiber contents varied and were either 150 or 200 lb of 0.025 in. diameter by 2.5 in. long fibers per cubic yard of concrete.

9.16.6 Jefferson, Iowa (Greene County). Another steel fiber concrete overlay was placed on a bridge on Highway E 53 near Jefferson, Iowa (Greene County) in September 1973.[109,308] The 160 ft long, 3 in. thick, overlay was fully bonded to the old bridge deck using a cement paste bonding agent and contained no joints. The fiber content was 160 lb of 0.010 in. by 0.022 in. by 1.0 in. long fibers per cu yd of concrete.

9.16.7 Winchester, Virginia. During June 1973, the State of Virginia constructed six new deck sections on two bridges on U. S. Route 7 bypass at Berryville, Virginia. A two-course, bonded construction technique was used in the deck construction. This involved the casting of an initial layer of concrete to a level near the top reinforcement and then following this with a high quality 2 in. thick wearing course. Steel fiber reinforced concrete was one of three materials being evaluated for this wearing course. Two test sections 52 ft by 38 ft were prepared using a concrete containing 752 lb of cement and 170 lb of 0.016 in. diameter by 1 in. long steel fibers per cubic yard of concrete. The performance of these decks will be compared to that of decks made with high quality portland cement concrete and latex modified concrete.

9.16.8 Rome, Georgia. In June, 1974 a bridge deck in Rome, Georgia, was overlayed with steel fiber reinforced concrete in an attempt to stiffen the bridge against the vibrations caused by the passage of heavily loaded gravel and lumber trucks. While the stiffness of the structure was not significantly improved, the performance of the overlay in accommodating the large oscillatory deflections of the deck without cracking has been extrordinary. After one year service very few cracks could be found in an overlay that visibly oscillated and damped out over the length of the deck with each passing truck.

9.16.9 Hudson, New York. During October, 1973, a steel fiber reinforced overlay was placed on Bridge No. 2 over the Roeliff Jansen Kill on New York State road 9-G near Hudson, New York. An existing asphalt overlay on the badly cracked concrete deck of this four-lane bridge was removed prior to construction of the overlay. The bridge sustains traffic from approximately 2300 vehicles a day of which 10 percent are trucks. Performance after one year has been excellent with no cracks occurring.[309]

9.16.10 Summary of Performance. All of the overlays described above except Cedar Rapids (Section 9.16.3) were either fully or partially bonded to the existing deck and most of these developed some cracks which in most cases have remained very tight and have not adversely affected the riding quality of the deck. The 3 in. thick unbonded overlay at Cedar Rapids was virtually crack free after 3 years of traffic.

9.16.11 Full Depth Decks and Orthotropic Decks. No full depth construction of fiber reinforced bridge decks has been attempted although an analysis of such a construction was made for the Pennsylvania Department of Transportation.[55] For a span of 7-1/2 ft, and using an allowable stress of 1000 psi in flexure, it was found that a bridge deck 6 in. thick made entirely of steel fiber reinforced concrete had the same strength as a standard 8-1/2 in. thick reinforced concrete deck.

Zollo[656] investigated the feasibility of steel fiber reinforced concrete overlays for orthotropic bridge deck type loadings by evaluating flexural specimens of composite beams consisting of (1) concrete cast over steel plates, and (2) concrete cast over floor grating. The concrete contained 2 percent by volume of 0.006 in. diameter by 1/2 in. long steel fibers. He concluded that the use of steel fiber reinforced concrete in composite with steel plates and gratings is a structurally flexible system that is highly crack resistant. Tests under static and fatigue loading suggested that the composite system exhibits substantial material toughness which is highly desireable for bridge deck wearing surfaces. Zollo[656] concluded that no appreciable reduction in design stress is necessary to account for fatigue loading conditions when fiber concrete is used. The nominal overlay on the plate and grating specimens was 1 in.

9.17 PAVEMENTS AND OVERLAYS

Fiber reinforced concrete pavements and overlays have been used in locations which are residential[152,650], rural,[284,407] urban,[25,26,152,153,158,197,260,262] industrial,[79,322,400,427,512] airports[*] and other specialized applications. The following sections review some of the more publicized of these many applications.

9.17.1 Residential. Two residential streets in Cedar Rapids, Iowa, were overlayed with steel fiber reinforced concrete during October 1972.[152,650] A 178 ft long by 28 ft wide (curb to curb) partially bonded, no joint, overlay was placed on Danbury Street, NE. The old 7-in.-thick reinforced concrete street was badly cracked and spalled. The preparation of the

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[*] 48, 152, 155, 188, 190, 192, 193, 249, 309, 321, 425, 426, 427, 539, 648

surface for the overlay involved simply broom cleaning and wetting. The thickness varied from 2-1/2 in. to 4 in. depending on the amount of deterioration of the underlying pavement. The concrete contained 846 lb of cement and 175 lb of 0.016 in. diameter by 1-in.-long steel fibers per cubic yard of concrete. The aggregate was 3/8-in. maximum size. A few months after placing, the overlay had a few cracks, but they were not opening up.[588]

The second placement was a 200-ft-long by 24-ft-wide partially bonded overlay placed on Fifth Avenue between 16th and 17th Streets in Cedar Rapids.[152,650] The old surface was brick covered with a partially removed asphaltic concrete overlay. The center of the street contained exposed rail tracks. The street received the same preplacement treatment as Danbury Street (above). The overlay thickness was 2.5 in. The concrete contained 846 lb of cement and 200 or 250 lb of 0.016 in. diameter by 1-in.-long steel fibers per cubic yard of concrete. Traffic was allowed on the finished overlay seven days after construction. Two transverse cracks developed early in the curing history over an old sewer construction area. Other than that, the performance has been judged excellent.[309]

9.17.2 Rural.

9.17.2.1 Ashland, Ohio. The first experimental project to evaluate the use of steel fiber reinforced concrete in new pavement construction was completed during August 1971, as the entrance to a truck-weighing station of Interstate 71, north of Ashland, Ohio.[407] The 500-ft-long by 16-ft-wide by 4-in.-thick pavement was placed, with no joints, on a 5-in. asphaltic concrete base. Ends were tapered to 9 in. over a 7-ft length. Doweled expansion joints were installed at either end. The concrete contained 846 lb of cement and 265 lb of 0.010-in. by 0.022-in. by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in.

The pavement was not officially opened to traffic for two years, but for more than one year it was used as a storage area and sustained an indeterminate amount of light and load bearing vehicles. The day following the placement, a transverse crack occurred 265 ft from one end or approximately at the midpoint of the slab.[407] The crack varied in width from 0.125 in. to 0.500 in. Four months later a second transverse crack occurred 100 ft from the end and 165 ft from the first crack. This crack only opened slightly. A third transverse crack occurred some time in 1973 and was 98 ft from the other end of the pavement. An irregular longitudinal crack, hairline in nature, was also observed over a 39-ft length at the center of the pavement beginning approximately 100 ft from the end of the pavement. All daily traffic at this site is truck traffic with approximately 2400 to 3600 trucks per day passing over the pavement.[407] This traffic produced spalling in the areas of the three transverse cracks and prompted the replacement of 15-ft sections in these areas with full-depth portland cement concrete.[309] The performance of this pavement suggested that future designs for this type of pavement include jointing details and reduced slab lengths.[407]

9.17.2.2 Greene County, Iowa. The most ambitious project built to date was an experimental fibrous concrete overlay project placed during September and October 1973, on a 3-mile section of Highway E53 near Jefferson, Iowa, in Greene County, Iowa.[284] The roadway selected to receive the fibrous concrete overlay was 3.03 miles of badly cracked pavement constructed originally in 1920 and 1921. The existing pavement was 18 ft wide and was being widened an additional 2 ft on each side. The total new width of 22 ft was overlayed. The project incluced thirty-three 400-ft by 20-ft sections of steel fiber reinforced concrete (2 and 3 in. thick), four 400-ft by 22-ft sections of continuously reinforced concrete overlays (3 and 4 in. thick), and five 400-ft by 22-ft sections of plain concrete and mesh reinforced concrete overlays (4 and 5 in. thick). Variables in the steel fiber reinforced concrete included:

a. Overlay thickness: 2 and 3 in.

b. Cement content: 600 and 750 lb per cubic yard.

c. Fiber contents: 60, 100, and 160 lb per cubic yard.

d. Fiber sizes: 0.010 in. by 0.022 in. by 1 in. long, and 0.025 in. diameter by 2.5 in. long.

e. Type and condition of bond with the old slab: bonded, partially bonded, and unbonded.

Joint spacing on the fibrous sections was 40 ft. The concrete used a 50/50 weight ratio of sand/coarse aggregate with the coarse aggregate being 3/8 in. maximum size. The amount of coarse aggregate used per cubic yard of concrete was greater than that used on any previous overlay project.

The Greene County project is the only experiment to date (1975) in which all of the major mixture and overlay design variables are being studied under the same loading and environmental conditions. The main shortcoming of the project is the light traffic count which is expected to be approximately 1000 vehicles per day. This will prolong the time required for a meaningful assessment of the performance of the overlays in service.[309] Early observations indicate that the use of debonding techniques has greatly minimized the formation of transverse cracks during the early life of the overlay. The 400-ft-long unbonded sections have exhibited fewer than two cracks (in some cases no cracks) after 9 months of service. Fully and partially bonded overlays have shown from 8 to 15 cracks per section.[309] The 3-in.-thick pavements were performing significantly better than the 2-in. pavements.[427] The 2-in. pavements were tending to deteriorate along the edges and joints for all bond conditions. These thin 2-in. overlays, when unbonded, were attempting to curl and warp. Bond was apparently being lost in the bonded sections. Cement content variations did not seem to affect performance but the 160 lb of fiber per cubic yard sections performed significantly better than the sections with only 100 or 60 lb per cubic yard of fibers.[427]

19.17.3 Urban.

19.17.3.1 Detroit, Michigan (Eight Mile Road). In October 1972, a steel fiber reinforced overlay was placed on a section of M 102 (Eight Mile Road) in Detroit, Michigan.[25,26,152,153,158] This was the first use of a fibrous overlay on a heavily traveled urban roadway. The original reinforced concrete pavement, constructed in 1930, was 20 ft wide and had a 12-ft concrete widening lane placed in 1955 and new 6- and 10-ft base course widening. The overlay was four lanes, 48 ft wide, with normal slab lengths of 50 and 100 ft. Two slabs were made approximately 67 and 79 ft long because of manhole covers. Pavement joints were sawed. The overlay was designed for a 3-in. minimum thickness and used concrete which contained 846 lb of cement and either 120 lb or 200 lb of 0.010-in. by 0.022-in. by 1-in.-long steel fibers per cubic yard of concrete. The aggregate used was 25A slag. The concrete was placed using a conventional batch plant and a slipform paver. The old concrete surface was simply swept and wet down prior to the placement of the overlay thus resulting in a partially bonded condition.

Heavy traffic in the area required opening the lanes two days after placement.[25] The construction was done under adverse weather conditions at curing temperatures considerably lower than desired. Through the first three months, performance of the 200-lb fiber content concrete sections was adequate, but the outside lanes of the 120-lb fiber content concrete sections developed some serious problems. Several reflective cracks developed over joints and cracks in the old pavement and closely spaced fractures developed in localized areas. In many locations, the planned overlay thickness was not achieved, with thicknesses occurring as thin as 1-1/4 in.[25] After nine months, the deterioration significantly progressed.[26] The overlay was determined to have become completely unbonded and some edge warping was occurring. Approximately 3/4 of the overlay was then removed. The problems were attributed to insufficient thickness and incomplete curing of the overlay prior to opening to traffic.[26]

9.17.3.2 Calgary, Alberta, Canada. Johnston[260,262] reported on the performance of plain and steel fiber reinforced test slabs placed directly on a granular subbase. The test section occupies a 180-ft length of one lane of an existing two-lane 24-ft asphalt concrete road which forms a city bus route through the University of Calgary campus in Canada. The test section is subjected to approximately 1000 loadings by busses each week. It consists of fifteen 12-ft-wide by 11-ft-long slabs ranging from 3 to 7 in. in thickness and containing 0, 67, or 133 lb of 0.01 in. diameter by 0.75-in. long steel fibers per cubic yard of concrete. The fibrous concrete mixtures used 500 lbs of cement and 200 lbs of fly ash per cubic yard of concrete and 1/2-in. maximum size aggregate. The plain concrete sections used the same aggregate but only 550 lb cement per cubic yard of concrete. Fiber contents were varied at 1/2 or 1 percent by volume. After 12 months observation, Johnston[262] concluded that for pavement of constant thickness, the 1 percent fiber content concrete has a life about four times that of the 1/2 percent fiber content concrete which in turn has a life about four times that of plain concrete. Also, the 1 percent fiber concrete has a life more than 12 times that of plain concrete. For equivalent performance in terms of cracking and, by implication, equivalent pavement life, the 1 percent fiber concrete can be used in pavements of less than 60 percent and possibly as little as 50 percent of the thickness needed for plain concrete. With 1/2 percent fiber concrete, the thickness could be reduced to 75 to 80 percent of that of a plain concrete pavement. He also noted that predictions of the performance of fiber concrete from relationships established for plain concrete using the standard flexural strength value as the measure of concrete quality are overly conservative.[262]

9.17.3.3 Great Britain (M10 Motorway). Gregory,[197] et al evaluated the potential of steel fiber reinforced concrete for thin overlays on heavily trafficked concrete roads in Great Britain. Overlays were constructed on a portion of a reinforced concrete carriageway on the M10 motorway. This motorway had been in service 15 years and had suffered some cracking. The overlay was constructed in two thicknesses, 2-3/8 in. and 3-3/16 in. using fiber contents of 1.3, 2.2, and 2.7 percent by weight of 0.02 in. diameter by 1-1/2-in.-long steel fiber. Sixteen test sections were placed in two lanes, some of which were bonded to the original pavement and some which were only partially bonded. The total length of the test overlay was 655 ft. Expansion joints were provided in the overlay at intervals of 120 ft to coincide with those in the existing pavement. Transverse contraction joints were formed at 40-ft intervals to provide test slabs of a reasonable length enabling all the variables to be covered. No dowelbars or tie bars were used in any of the overlay joints.

One week after placing and prior to opening the overlay to traffic, some hairline cracking developed in one lane and this was attributed to shrinkage and thermal movements.[197] Numerous reflection cracks also occurred and appeared to be independent of whether the overlay was bonded or partially bonded. Traffic was allowed on the overlay after it was from 7 to 11 days old. After 11 days of traffic, numerous reflection cracks had occurred along with a few new cracks. Most remained as hairline cracks. The total length of longitudinal cracking in the overlay exceeded that of the original pavement and longitudinal cracks tended to be more pronounced in the partially bonded sections. Between 11 weeks and 6 weeks after opening the road to traffic, there was little further change in the crack pattern. Gregory, et al[197] found no clear relationship in their work between fiber content and resistance to cracking but concluded that the thinner bonded sections performed better than the others.

9.17.4 Industrial.

9.17.4.1 Niles, Michigan. Luke, et al[322] have described a number of full thickness slabs and overlays of steel fiber reinforced concrete placed at Niles, Michigan. The first full thickness slab was placed in September 1968 and was 4 in. thick. It was placed as one lane of a road into an industrial plant. The other lane was 7-1/2 in. of conventional concrete. After four years and eight months, the conventionally reinforced concrete lane had three transverse cracks across the complete width. The steel fiber concrete section had no cracks even though trucks routinely cut the corners of this lane. The steel fibers used were both flat and round being 1 in. long with a 0.012 in. diameter or equivalent diameter. In June 1970, a section of road 12 ft wide and 160 ft long leading out of a ready-mix concrete plant was then constructed using steel fiber reinforced concrete. The road would carry the ready-mix trucks at axial loads of 16,000 and 18,000 lbs with a total weight of 50,000 lb passing over the road each day.

The road was broken down into 13 test sections of different lengths, thickness, mixture proportions, fiber reinforcements, and different laminations and overlays.[332] Test section lengths varied from 2 ft to 20 ft with slab thicknesses varying from 2 to 4 in. Overlays and toppings were 2 in. thick. Fiber contents were varied at 185 or 200 lb per cubic yard of concrete. Fiber dimensions included diameters of 0.010, 0.015, and 0.016 in. and lengths of 3/4 and 1 in. Loads were allowed on the new concrete 64 hours after placement. After 2 years, the only section containing no cracks was 4 in. thick and contained 200 lb of 0.016-in. diameter by 1-in.-long fibers per cubic yard of concrete. All sections made at thicknesses of 3 in. or less experienced varying degrees of cracking with most of it beginning after one year of service. A 2-in. overlay on 6 in. of conventional concrete had only 3 hairline cracks after 2 years.

9.17.4.2 Japan (Kashima Works). Nishioka, et al[400] reported on the use of a steel fiber reinforced pavement in the Kashima Works yard of Sumitoma Metals in Japan. The road was routinely used by fork lift trucks with a gross weight of 52 tons and the heavy wheel loading badly damaged conventional pavements. A steel fiber reinforced pavement for similar types of loadings was also constructed at the Homestead Works of US Steel in Pittsburgh, Pennsylvania, in 1969.[512] The Japanese pavement was done in two sections each 50 ft long, 13.6 ft wide, and 6 in. thick. The joint interval was varied from 16.4 ft in one section to 49.2 ft in the other section. An 8-in.-thick conventional pavement with a 16.4 ft joint interval was also placed as a control section. The concrete had a cement content of 375 lb per cubic yard of concrete, a water-cement ratio of 0.464 and a steel fiber content of 222 lb of 0.010 in. by 0.022 in. by 1-in.-long fibers per cubic yard of concrete.

Loading tests were performed one month after the concrete was placed with slab stresses being determined from strains obtained from molded wire strain gages.[400] The maximum bending stresses measured on the surface at the corner edge of the slab at the point of severest loading were 545 and 480 psi, respectively, for the steel fiber concrete and the conventional concrete. The measured flexural strength of both these concretes was 995 and 650 psi, respectively. An examination of the cracking pattern in the slabs after one year showed that in the conventional concrete slab, there was a large crack which went through the thickness near one corner and 10 rather small cracks of 6 to 16 ft in length. There was only one hairline crack in the center of a section of fiber reinforced pavement with the 49.2-ft joint spacing. This was attributed to shrinkage and the long joint spacing. Nishioka, et al[400] concluded that the joint interval of steel fiber reinforced concrete pavement could be longer than that of conventional concrete, and that the life of the pavement became remarkedly increased in proportion to the volumetric percentage of reinforcing fibers.

9.17.4.3 Midlothian, Texas (Gifford-Hill Quarry). Buckley[79] evaluated four test slabs containing glass fibers and one control slab containing steel reinforcing bars on a road used at a quarry of the Gifford-Hill Cement Company, Midlothian Division. Each slab was 16 ft wide and 20 ft long placed on a subgrade whose modulus of subgrade reaction "K" was conservatively estimated to be 400 kips per square in. The slabs contained the following variables:

Slab 1. 8-in.-thick, reinforced with No. 3 bars on 24-in. centers.

Slab 2. 6-in.-thick, glass fiber reinforced.

Slab 3. 6-in.-thick, glass fiber reinforced.

Slab 4. 4-in.-thick, glass fiber reinforced.

Slab 5. 4-in.-thick, glass fiber reinforced on 1 in. of recycled rubber.

The concrete for the glass fiber reinforced slabs used 1-in.-long glass fibers, 564 lb of cement per cubic yard of concrete, 1/2-in. maximum size aggregate, sand, and water. Glass fiber contents varied from 1.34 volume percent for slab 5 to 0.97 volume percent for slabs 2, 3, and 4. The water-cement ratios also varied being 0.800, 0.688, 0.688, and 0.700, respectively, for slabs 2, 3, 4, and 5. The control slab used 470 lb of cement per cubic yard of concrete, a water-cement ratio of 0.65, and 1-1/2-in. maximum size aggregate.

Loads were applied by a Euclid rock hauler which, when loaded, carried a front axle load of 43,000 lb or 21,550 lb per front wheel and a rear axle load of 85,200 lb or 21,300 lb per rear wheel. Buckley[79] used a cracking index C' where

$$C' = \frac{\text{crack length}}{1000 \text{ ft}^2 \text{ of surface}} \tag{9.19}$$

to describe the relative performance of each slab. The results are shown in Figure 9.36. Only the 8-in. glass fiber reinforced slab performed better than the conventionally reinforced slab. The other slabs had become so badly cracked they had to be removed. After one year's observation, minor growth of the crack patterns was continuing with the glass reinforced concrete slab performing somewhat better than the conventional reinforced slab.[79]

9.17.4.4 Vicksburg, Mississippi, (Waterways Experiment Station). Two experimental pavements were placed on a street at the Waterways Expriment Station (WES), Vicksburg, Mississippi, in 1972 and 1973. The first of these was placed in June 1972 and was 100 ft long, 20 ft wide, and 5 in. thick placed on rough graded soil. The slab contained no joints and was placed using transit-mix trucks, hand labor, and a vibrating screed. The concrete used 822 lb of Type I-P cement (containing approximately 15 percent fly ash) and 115 lb of 0.016-in. diameter by 3/4-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in. After four years of service with the first year subjecting the pavement to heavy truck and construction equipment loads and the later years involving only light truck and car traffic, the slab remains crack free.

The second WES pavement was constructed in July 1973 and was 1000 ft long, 24 ft wide, and 4 in. thick.[427] It was constructed as one 24-ft-wide lane without joints using slipform paving techniques. The pavement was permitted to crack where necessary to reduce tensile stresses induced during curing. The ends of this pavement were adjacent to the fiber slab described above and a new 6-in. concrete pavement. A 1-in. expansion joint was provided at both ends of the pavement. The pavement was placed on a clay-gravel subbase. In places, the grade of the placement approached 12 percent and this presented a few problems with the slipform paving operation proceeding downhill. The concrete mixture used 846 lb of cement and 200 lb of 0.010 in. by 0.022 in. by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in.

Immediately after placing some shrinkage cracking or crazing occurred in this second pavement because application of curing compound did not keep up with the paver. The problem was accentuated by high ambient temperatures (95F), the high cement content (846 lb per cubic yard), the rough finishing technique, and the downhill paving. A distinct crack pattern developed in the pavement the night after placing. Seven cracks ranging in spacing from 63 to 240 ft occurred. The average length between cracks was 126.4 ft. The cracks widened considerably more over a period of a few days and then stabilized except for normal thermal opening and closing. These cracks from the inception were not hairline cracks held tightly closed by fibers such as is the case for load-induced cracks. No additional cracking has been observed in the pavement. The traffic load is light, however, but does include an occasional truck loading.

9.17.5 Airports. Seven airport uses of steel fiber reinforced concrete* and four experimental test slabs for aircraft type loadings[188,192,193,427] have been reported. All of the airport uses except one small slab[321] have been overlays. The experimental slabs involved both full depth slabs and overlays.

9.17.5.1 Vicksburg, Mississippi (Waterways Experiment Station). Two experimental fibrous concrete test pavements consisting of a slab-on-grade and an overlay were constructed at WES during June 1971-March 1972 as part of a test section designed to study the effects of multiple-wheel heavy gear load aircraft such as the Boeing 747 and the military C-5A transport on keyed longitudinal construction joints.[188,192,193,427] The steel fiber reinforced slab on grade was located at the east end of a 320-ft-long test track. The slab was 25 ft long and 50 ft wide resting a 4-in.-thick sand filler course having a modulus of subgrade reaction

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*References 152, 309, 321, 425, 426, 427. Also see notes on pages of 42 pci. The 6-in. slab was thickened to 9 in. by a uniform taper over 30 in. at the transverse edges. No provision for load transfer to other adjacent slabs was made. A plain concrete nonreinforced test section 50 ft square (4 to 25 ft square slabs) and 10 in. thick was also constructed on a 4-in.-thick sand filter course having a modulus of subgrade reaction of 125 pci and was used for making comparisons of performance between the fibrous concrete and conventional concrete pavements. The concrete for the slab used 846 lb of cement and 250 lb of 0.016-in. diameter by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in. The construction of both slabs used manual techniques because of their small size. The steel fiber reinforced concrete slab contained no joints.

Loads were applied 73 days after placing the concrete by using a WES load cart which simulated one 12-wheel gear loading with 30,000 lb per wheel. The cart was driven back and forth along five evenly spaced parallel lines across an area 200 in. wide on the pavements.

The 6-in.-thick fibrous concrete slab on a weak subgrade is about half the design thickness of the 10-in.-thick plain concrete slab on a medium strength subgrade. The 6-in-thick fibrous concrete slab developed its first visible crack at 350 traffic loadings and the second visible crack at 700 traffic loadings. The 10-in.-thick plain concrete slab developed the first crack at less than 40 traffic loadings and was in a shattered condition after 700 loadings. After 950 loadings this pavement was completely failed due to structural cracking and spalling. The progression of cracking in the fibrous slab was gradual. At 8735 traffic loadings, many hairline-width cracks had developed but the pavement was in excellent condition.

The completely shattered plain concrete pavement was then cleaned and moistened and covered with a no-joint fibrous overlay 4 in. thick. The overlay concrete was similar to that of the fibrous slab except that the steel fibers were 0.010 in. by 0.022 in. by 1 in. long in size. The overlay was loaded in the same manner as the slab and was 28 days old when loading began. The first crack formed at 900 traffic loadings and was a reflection of the longitudinal joint in the base pavement. The second and third cracks appeared at 1400 and 2600 loadings, respectively. Other cracks tended to form gradually under traffic. After 6900 traffic loadings, only one crack was classified as a working crack.[193]

Between March 1972 and May 1973, two additional fibrous concrete test sections were constructed and tested at WES as a part of a series of full-scale pavement tests designed to study the effects of chemical stabilization, insulating materials, and fibrous concrete on pavement performance.[427] The first pavement consisted of 7 in. of steel fiber reinforced concrete on a 20-in. layer of lean clay encased in a waterproof membrane over a heavy clay subgrade. The second pavement consisted of 4 in. of the same concrete on a 17-in. layer of clay gravel stabilized with 6 percent portland cement over a heavy clay subgrade. Each pavement was composed of two 25-ft by 50-ft slabs divided by a construction joint. The concrete used three different fibers. Round, 0.016-in. diameter by 1-in.-long fibers were used for the full depth pavement. One-half of the second pavement used 3/4-in.-long deformed fibers having a 0.016-in. diameter while the other half used flat fibers of 0.010 in. by 0.014 in. by 3/4 in. long cross-section. Test traffic was applied in a similar manner as that described above for the first two fibrous concrete test sections except that both a 200-kip and 240-kip twin-tandem assembly equipped with 49 x 17 tires with a ply rating of 26 were used. These represented one twin-tandem component of a Boeing 747 assembly.

One lane of the full thickness, 7-in. steel fiber reinforced concrete slab initially failed after 1000 coverages of the 200 kip, dual tandem assembly. Although the slab was not completely bisected by a crack, it was felt that the transverse crack and permanent deformation along the longitudinal construction joint was as detrimental as a continuous crack across the slab. The pavement was considered to have reached the shattered slab condition at 1800 coverages. Complete failure occurred at 3000 coverages. The other lane of this pavement was loaded using the 240-kip, dual-tandem assembly. Initial failure in this case occurred at 200 coverages. The slab was considered completely shattered at 650 coverages and completely failed at 1010 coverages. Failure of the pavement under both 200- and 240-kip traffic was characterized by multiple cracking.

The cracks did not spall but did ravel around the edges and widen as traffic was applied. The pavement also experienced a maximum permanent deformation of approximately 0.7 in. in both lanes.

The 4-in.-thick pavement over the clay-gravel and cement stabilized clay subgrade was loaded in the same manner as the 7-in.-thick pavement. Under the 200-kip loading, initial failure of the first lane occurred at 500 coverages. At 1200 coverages, it had reached the shattered slab condition. Some of the longitudinal cracks had begun to widen and transverse cracks connecting the longitudinal cracks were beginning to occur. Complete failure occurred at 1770 coverages. When the second lane was loaded with the 240-kip, dual-tandem assembly, initial failure was noted at 150 coverages. Permanent deformations of 0.5 in. had also occurred at this point. The shattered slab condition occurred at 400 coverages with complete failure occurring at 740 coverages. As in the case of the 7-in. slab, the 4-in. slab failure was characterized by multiple cracking and large permanent deformations. It was noted that the pavements behaved more like flexible than rigid pavements.[427]

9.17.5.2 Tampa, Florida. During February 1972, two fibrous concrete overlay test pavements were constructed at Tampa International Airport, Florida.[426] The overlays were constructed on a taxiway parallel to one of the primary runways. The existing taxiway pavement was constructed in 1965 and opened to traffic in January 1966. Two thicknesses of overlay were used. A 4-in. overlay was situated in the center of the taxiway and was formed by the construction of two 25-ft-wide paving lanes. The paving lanes were constructed so that the center of the paving lanes coincided with the longitudinal construction joints in the base pavement. The longitudinal construction joint was a butt-type joint without load-transfer capabilities. No transverse joints were formed in the overlay. The ends of the section coincided with transverse contraction joints so that one transverse contraction joint and one longitudinal construction joint in the base pavement were spanned by each lane of the overlay. Bituminous transition overlays were constructed around the overlay.

A 6-in. overlay spanned the entire width of the taxiway. The section was formed by constructing three 25-ft-wide paving lanes. The longitudinal construction joints in the overlay matched the longitudinal construction joints in the base pavement. Vertical faces were formed along the edges of the paving lanes with no provisions for load transfer. No transverse joints were formed in the overlay; therefore, all transverse contraction joints in the base pavement were spanned by the overlay. One expansion joint in the base pavement located 75 ft from the south end of the section was spanned by the overlay. The ends of the overlay coincided with transverse contraction joints in the base pavement. Bituminous transition sections were constructed on each end of the overlay.

A minimum of surface preparation was performed, and no attempt was made to either bond the fibrous concrete to the base pavement or to completely eliminate the bond between the overlay and base pavement. The concrete used 517 lb of cement, 225 lb of fly ash, and 200 lb of 0.010 in by 0.022 in. by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/4 in.

These overlays were partially bonded but no reflection cracking was observed prior to opening for traffic. After traffic was applied, reflection cracking occurred. The cracks in the overlays coincide with cracks or joints in the base pavement. Those cracks corresponding to base pavement joints have widened sufficiently to cause failure (bond or fracture) of the fibers across the cracks. Those cracks corresponding to base pavement cracks have not widened significantly thus indicating that the fibers across the crack are still effective.[427]

9.17.5.3 Lockbourne AFB, Ohio. Two slabs were placed at Lockbourne AFB, Ohio, in July 1970.[427] The first slab was a paving apron on grade that was approximately 35 ft by 46 ft by 6 in. thick with a 4-in. by 5-in. leave out in the center. The second slab was approximately 5 ft by 22 ft by 6 in. thick placed on grade on a taxiway. The slabs were constructed over 9 in. of a lean concrete (376 lb cement per cubic yard) and granular base course. The fiber reinforced concrete contained 725 lb of cement and 180 lb of 0.010 in. by 0.022 in. by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in. The 6-in. fibrous concrete slabs and the 9-in. lean mixture concrete was separated by 4-mil polyethylene sheeting thus producing an unbonded overlay.

Cracks developed at each corner of the leave-out in the parking apron but were arrested in growth by the fibers at approximately 3 to 4 ft from the corners. Without the fibers, the cracks probably would have propagated to a free edge.[427] No distress was observed in the taxiway slab, but an adjacent 25 ft by 12 ft by 15-in.-thick plain concrete slab had developed a longitudinal crack and a number of short transverse cracks.

9.17.5.4 Cedar Rapids, Iowa. In September 1972, a steel fiber reinforced concrete overlay was placed on the main taxiway at the Cedar Rapids, Iowa, airport.[152] Deteriorated concrete and asphalt patches in the old taxiway were removed prior to the overlay operation. The overlay was 86 ft long, 75 ft wide, and varied in thickness from 1 to 4 in. across the width. The base concrete had longitudinal joints at 12-1/2-ft intervals and transverse joints spaced at 20 ft. The overlay pavement had no joints except a longitudinal construction joint in the center. It was placed using a Fomaco bridge machine. A concrete containing 752 lb of cement and 200 lb of 0.016-in. diameter by 1.0-in.-long steel fiber per cubic yard of concrete was used for the east half of the overlay. The same mixture but with 150 lb of 0.025 in. diameter by 2.5-in. long steel fibers was used for the west half.

The completed overlay was opened to aircraft traffic seven days after construction. At this time the overlay had reflective cracks from the longitudinal construction and contraction joints and transverse construction joints in the base pavement.[427]

9.17.5.5 Detroit, Michigan. A steel fiber reinforced concrete aircraft parking apron slab (on grade) was placed in July 1971 at the Detroit airport.[321] One of the problems with pavements at this airport was the drain boxes around which conventional portland cement concrete continues to crack providing a repetitive repair problem. The slab placed was in a gate area used by 747 aircraft. It was 20 ft by 30 ft by 8 in. thick and used concrete containing 0.016-in. diameter by 1-in.-long fibers. The adjacent slabs were 12 in. thick so the base course for the steel fiber reinforced concrete slab had to be built up

4 in. The fibrous slab was tied to adjacent slabs with deformed rebars installed by drilling and grouting in adjacent slabs. Observations at nine months indicated that the slab was performing satisfactorily.[321]

9.17.5.6 New York, New York (JFK Airport). During May 1974, an unbonded steel fiber reinforced concrete overlay was placed on the end of a main runway at John F. Kennedy (JFK) International Airport, New York, New York.[309] Construction consisted of the removal of the old asphalt concrete wearing course, construction of a new 2-in.-thick asphalt leveling course, placement of a double thickness of 6-mil thick polyethylene, and then construction of the steel fiber reinforced concrete overlay. The overlay was 175 ft by 120 ft by 5-1/2 in. thick and used both keyed and doweled construction joints. The concrete contained 752 lb of cement and 175 lb of 0.025-in. diameter by 2.5-in.-long steel fibers per cubic yard of concrete. Observations of pavement performance have not yet been reported.

9.17.5.7 Las Vegas, Nevada. The largest fibrous concrete paving project to date was done at the Las Vegas McCarran International Airport in early 1976 with the placement of a 945-ft by 600-ft by 6-in.-thick overlay on a transit parking apron and connecting taxiway.* A problem for overlaying the existing pavement existed because existing cargo buildings were already 2 to 3 ft below the ramp. A conventional concrete overlay would add 15 to 16 in. to this difference. The selection of fiber reinforced concrete allowed this thickness to be reduced to 6 in.

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*Recent References (not in bibliography):

a. "Fibrous Concrete Overlay at Las Vegas Airport," American Concrete Paving Association Newsletter, Vol 12, No. 6, Jun 1976, p 2.

b. "Fibrous Concrete Cuts Airport Overlay to 6 In.," Engineering News Record, Vol 196, No. 24, 10 Jun 1976, p 21.

The overlay was placed in alternating 25-ft strips using slipform paving techniques. Construction joints were spaced 50 ft apart. The concrete mixture contained 160 lb of 0.010 in. by 0.022 in. by 1-in.-long steel fibers per cubic yard of concrete. The pavement was placed in 20 working days. The overlay was designed for a 20-year service life.

9.17.5.8 Reno, Nevada. A steel fiber reinforced concrete overlay was completed in May 1975 at the City of Reno Airport.* The 4-in. overlay was placed on the hardstand at the passenger debarking area at the air terminal building. This involved approximately 35,000 sq yd of surface. The concrete contained 658 lb of cement, 216 lb of fly ash, and 200 lb of 0.010 in. by 0.022 in. by 1-in.-long steel fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in. The concrete was initially spread to grade with a Blow-Knox spreader, vibrated with a mobile three-bar roller-vibrator, dressed with bull floats, scored with a rake, and then sprayed with curing compound. Sawed joints and grooves were placed in the overlay directly above joints and grooves in the original paving. The original paving had joints forming 20 ft by 25 ft slabs with a groove cut down the center of the 25-ft side thus resulting in 12-1/2-ft by 20-ft subsections. Information on the performance of this pavement has not yet been made available, although the author understands that some curling and warping of these small sections has occurred.

9.17.6 Other Pavement Applications.

9.17.6.1 Fort Hood, Texas (Tank Parking Area). The first full-scale nonexperimental steel fiber reinforced concrete overlay was completed in March 1974 at Fort Hood, Texas.[631, 634] The area overlayed is a maintenance hardstand for tanks, tank retrievers, and other tracked vehicles. The existing pavement consisted of 5 to 7 in. of asphaltic concrete on a stabilized limestone base. Because of severe wear and rutting caused by the action of the tracks, the flexible pavement surface has required

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* Denson, R. H., "Report of Trip to Reno, Nevada, to Observe Fiber-Reinforced Concrete Paving Project," Memorandum for Record, Waterways Experiment Station, Concrete Laboratory, Vicksburg, MS, 8 May 1975, 2 pp.

replacement every three to four years. The steel fiber concrete overlay was selected in hopes to extend that maintenance period.

The concrete contained 519 lb of cement, 231 lb of fly ash, and from 160 to 200 lb of 0.010-in. diameter by 0.50-in.-long fibers per cubic yard of concrete. The maximum aggregate size was 3/8 in. Approximately 32,000 sq yd of 4-in.-thick overlay were machine placed in 20-ft-wide lanes with several thousand additional square yards being hand placed. Joints were sawed every 50 ft. During an early evaluation, only a minimum amount of cracking had appeared in the overlay.[634] This occurred a few days after placement and before traffic was allowed on the overlay. Only one shrinkage crack 8 to 12 ft long had appeared on the several thousand square yards of overlay placed by hand. Several transverse shrinkage cracks appeared in the machine-placed areas, but the most common crack was a longitudinal one in the center of the paving lanes. Several hundred feet of this type of cracking occurred for undetermined reasons. All the cracks were hairline and none appeared to be working cracks after the pavement was open to traffic.

A survey of the overlay after being opened to traffic for nine months revealed only one working crack.[631] It crosses a 12-ft-wide hand finished lane and has opened up approximately 1/4 in. The hairline cracks that developed shortly after construction remained tightly closed. The overall performance of the overlay has been very satisfactory.[631]

9.17.6.2 Patching. Luke[321] reported on 11 small patches that were installed along the key joint on a runway used by 747's at Chicago's O'Hare International Airport in October 1970. The runway was 12 in. thick and had cracks across it about 5 ft. The runway was reinforced with reinforcing steel every 6 in. in both directions of a depth of 6 in. The patches were cut out with a saw and an air hammer was used to remove the material. They were approximately 1 ft wide by 3 to 11 ft long and 3 to 6 in. deep and were blown out with air, wetted, and filled with steel fiber reinforced concrete containing 0.016 in by 1 in. round fibers. Similar repair work two years prior to then had been done with epoxy and not proved satisfactory. Luke[321] reported the steel fiber reinforced concrete patches were performing satisfactorily after two winters.

Walker and Lankard[612] reported on the use of precast steel fiber reinforced mortar slabs for patching the Prospect Expressway, South[612] and the Queens Midtown Tunnel in New York, New York.[614] The Queens Midtown Tunnel patching was done in March 1971. The pavement in the truck lane of the westbound tube had several areas where the brick surface had settled as much as 3 in. Repair of these areas required removal of a 3-1/2-ft square of the brick surface and the deteriorated concrete subbase. The repairs had to be completed between the hours of midnight and six a.m. of the same day. The precast slabs were 3 ft square and 2 in. thick. The excavation was approximately 7 in. deep and was filled with freshly mixed steel fiber reinforced mortar to a depth of 5 in. The precast slabs were then set into the fresh mortar and adjusted to be flush with the adjacent wearing course. The void between the precast slab and the adjoining bricks was then filled with the freshly mixed fibrous mortar. The mortar for the slabs contained 972 lb of cement and 245 lb of 0.010 in. diameter by 1-in.-long steel fibers per cubic yard of concrete. The subbase concrete mixture contained 808 lb of cement and 193 lb of 0.016-in. diameter by 1-in.-long steel fibers per cubic yard of concrete, and a set accelerator. Traffic was allowed on the patches approximately four hours after the concrete was placed.

In October 1971, a similar technique was used to repair an area in the extreme right lane between 8th Avenue Exit and the Fort Hamilton Parkway on the Prospect Expressway South. A 40-in. square of deteriorated concrete pavement was removed by saw cutting to a depth of 5 in. and removing the material with air hammers. The section was approximately 12 in. from the curb and abutted an expansion joint which was not replaced. The excavated patch area was filled to a depth of approximately 3 in. with a proprietary fast setting concrete mixture, the precast steel fiber reinforced mortar patch set in place and positioned, and the edge voids filled with the fast setting mixture. Traffic was resumed on the slab approximately 45 minutes after the work was done.

9.17.7 Summary of Performance. The majority of the pavements and overlays placed to date have had some visual examination to qualitatively assess their performance. Meaningful evaluations of performance have been made difficult by the relatively brief observation periods and the fact that only one project, Greene County, Iowa, studied major design variables systematically under identical load and environmental conditions. Even with only limited information available, some design procedures for both pavements and overlays have developed.[346,427,471,472,473] The analysis of the data has been based on several rather arbitrary, though conservative, decisions and assumptions. The existing design criteria should be considered as tentative and subject to change as additional performance data are accumulated.

Fibrous concrete pavements have generally performed better than comparable plain concrete pavements having identical thickness, foundation conditions, and concrete flexural strength. The use of fibrous concrete will result in thinner pavements and offer an alternative design that has several advantages over conventional construction.[427]

Many of the experimental overlays placed to date have developed full-width transverse cracks within 24 to 36 hours after placing. Lankard and Walker[309] felt that this problem was related to the high cement content and relatively low aggregate content of the fibrous concrete used in the experiments. The drying shrinkage and heat release of these concretes is greater than for conventional concretes. Restrained shrinkage occurs in the overlay at a time when the bond between the fiber and matrix is inadequate and a crack is formed. They noted[309] that this situation was at least partly responsible for the need to remove spalled portions of overlays at Detroit and Ashland, Ohio. Successful techniques for eliminating or minimizing this problem have included: (1) reducing the cement content and increasing the aggregate content of the concrete; (2) replacement of a portion of the cement with fly ash; and (3) using a shrinkage compensating concrete.

Slab movements due to temperature change must also be accomodated. The use of double polyethylene sheeting as a bond breaker has been very successful in many of the field projects. Good examples are the unbonded sections at Greene County and Cedar Rapids, Iowa. Experience indicates that some form of debonding will be necessary in steel fiber reinforced concrete overlays to eliminate nonload related cracking, especially when overlaying existing concrete pavements.

The use of sliding dowels to join adjacent fibrous concrete overlay slabs or to joint fibrous concrete overlay slabs to existing pavement has been successful in sections as thin as 3 in.[309]

9.18 EASE OF CONSTRUCTION

The use of fiber reinforced concrete in pavements and overlays is more difficult than using plain concrete. The first area of difficulty occurs in the mixture proportioning. Typical steel fiber reinforced concretes used in pavements and overlays have contained proportions in the following ranges:

Cement content or cement plus fly ash	500 to 950 lb/yd^3
Water-cement ratio (by weight)	0.35 to 0.60
Coarse aggregate as a percentage of total aggregate content	25 to 50 percent
Maximum size coarse aggregate	3/8 to 3/4 in.
Fiber contents (by volume)	0.5 to 2.0 percent
Fiber lengths	1/2 to 2.5 in.
Fiber diameters	0.010 to 0.025 in.

Of first note is the high cement and sand contents. This is caused by the addition of the fibers. Two hundred pounds of 0.016-in. diameter by 3/4-in.-long fibers in a cubic yard of concrete occupies approximately 711 cubic inches of absolute volume based on an assumed specific gravity of 7.8 (See Table 9.1). Each individual fiber occupies 0.000151 cu in. of volume and has a surface area of 0.0377 square inches. This means there are approximately 4.7 million fibers in a cubic yard of concrete or approximately 1234 sq ft of fiber surface area which must be wetted by mortar in order for bond to the fibers to develop. Hence, both cement and sand must be increased to provide the mortar.

With this large number of fibers, the problem of interference with aggregate particles and its adverse effect on workability has developed. With very few exceptions, all of the pavement and overlays have been constructed using 3/8-in. maximum size aggregates. The use of fly ash in the mixtures has been beneficial in improving the workability. The large number of fibers has also resulted in the balling or clumping of fibers both during batching and mixing. When this occurs, the fibers are bunched together and are not distributed throughout the pavement. The balls or clumps also act as stress raisers and can precipitate localized cracking. The clumping during mixing can usually be solved by procedural methods relating to the sequence of materials batching. The proper sequence will have to be established for the actual equipment to be used. The formation of balls or clumps during batching must be eliminated before the balls or clumps go into the mixer as generally the mixing action will not break them up. This has been done using a wide variety of techniques and equipment which have been more or less successful. However the problem is solved, it will still involve an additional piece of equipment during batching that is not normally used for plain concrete.

Additional labor will be required at the batching facility to introduce the fibers into the mixture. Presently all steel fibers are shipped in 40 to 50-lb cardboard boxes which must be opened and the fibers removed. This removal usually consists of dumping the fibers into some appropriate separating device which then discharges the fibers into the batching or mixing system. A 10 cu yd central batching plant usually produces 147 cu yd of concrete an hour or approximately one batch every 4 minutes and 5 seconds. Of this time 20 seconds is allowed for batching with the rest for mixing and discharging. To introduce 2000 lb of fibers to a 10-cubic-yard batch of concrete (200 lb per cu yd fiber content) would require the handling of fifty 40-lb boxes. This will take considerably longer than 20 seconds and even with a large labor force perhaps as long as five minutes. The total batch production time will probably be doubled thus reducing the plant capacity by one-half.

The placing of steel fiber reinforced concrete must be exact, as it is almost impossible to move this material by mechanical means or vibration once in place. Shovels for hand labor must be replaced by rakes and forks. Finishing is not significantly different than for normal concrete except that a burlap drag cannot be used. The burlap snags the fiber and pulls them out of the mixture thus causing disruptions in the surface. All labor should be required to wear heavy gloves and safety goggles to avoid puncture by the fibers.

Curing is the same for fibrous concrete as for plain concrete. Joint sawing and grooving can be done in the conventional manner with standard equipment.

9.19 EASE OF MAINTENANCE

The removal of fibrous concrete using conventional breaking equipment is complicated by the excellent energy absorption characteristics of the material. Concrete jack hammers are not effective as the fibers tend to keep the concrete from breaking apart. Drop hammers are also not effective. Chipping machines have worked but the progress was slow. The most effective means of removal has been by getting under the edge of the concrete and essentially lifting and rolling it up. In general, considerably more effort will have to be expended to remove a fibrous concrete section than a plain concrete or reinforced concrete section.

9.20 ENVIRONMENTAL COMPATIBILITY

The use of fibrous concrete in pavements should not present any additional environmental problems than would the construction of a plain concrete pavement. Concern has been expressed from time to time on the potential for fiber protrusion from the pavement. Following final texturing, some fibers may be exposed on or near the wearing surface of the pavement or overlay. Almost without exception, all of these fibers have been observed to be in the plane of the overlay and have not caused problems from the standpoint of tire penetration. Normal weathering and traffic cause most of these surface fibers to be removed. The very small cross-sections allow them to rust away very rapidly.

9.21 AVAILABILITY AND COST

The availability of steel and glass fibers is very good while it is very limited for other types of fibers. Presently there are four producers of steel fibers in the United States. The glass fiber market has directed itself towards nonstructural applications although the fibers are available for structural purposes. The steel fiber market has been concerned mostly with pavement applications and for the most part has had no trouble in meeting the supply demands of the paving industry.

The unit cost of fibrous concrete will be substantially more than that of plain concrete. The average cost of steel fibers is approximately $0.25 to $0.30 per lb f.o.b. the factory. At a fiber content of 200 lb per cubic yard of concrete the basic materials cost increase would be $50 to $60 dollars per yard plus the additional cost for cement as higher cement contents are used. The increase in cost when glass fibers are used is not expected to be significantly different that that for steel. When making cost comparisons of fibrous and plain concrete for pavements, the unit cost for materials should be adjusted to take into account that similar pavement performance can be obtained with reductions of pavement or overlay thickness (hence less materials required) and also reduced frequency of joints. Johnston[260] noted that the difference between the cost of plain concrete and fibrous concrete depended on the relative magnitude of four independent factors:

a. Costs directly associated with fiber.

b. Material and labor costs for placing concrete (plain or fibrous).

c. The reduction in pavement thickness made possible by inclusion of fibers (30 to 50 percent has been reported).

d. The reduction in joint costs made possible by the inclusion of fibers (not yet clearly defined but somewhere between 1/2 and 1/8 as many joints have been reported).

The first two are essentially fixed by local prices and wages and geographical proximity to a source of fibers. The last two are essentially a function of fiber characteristics and anticipated loadings.

Not much actual cost data have been published. Gramling and Nichols[186] reported that the construction of a steel fiber reinforced concrete overlay for a bridge deck in Pennsylvania in 1972 was awarded at a bid price of $215 per cubic yard for in-place concrete (all preparation work included). This averaged out to $25 per square yard of surface for a 2-in.-thick overlay. Another bridge deck overlay in Michigan in 1972 reported[148] a cost per square yard of $9.28 per square yard for a 3-in.-thick overlay. The concrete cost here was $70.30 per yard. The Fort Hood overlay, placed in 1974, had a steel fiber concrete cost of $83 per cubic yard of concrete.[634] More recently, the 6-in. overlay placed at McCarran International Airport in Las Vegas had a cost of $16.33 per square yard in place.[*]

[*] "Fibrous concrete cuts airport overlay to 6 in.," Engineering News Record, Vol 196, No. 24, 10 Jun 1976, p 21.

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CHAPTER 9

FIBER REINFORCED CEMENT AND CONCRETE

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Table 9.1

Typical Properties of Fibers

Type of Fiber	Typical Diameter in. × 10^{-3}	Maximum Fiber Length (in.)	Specific Gravity	Tensile Strength psi × 10^3	Elastic Modulus psi × 10^6	Elongation at Break Percent
Steel	0.2-20	As desired	7.8	40-600	22-30	3-4
Glass	0.35-0.8	As desired	2.5-2.7	300-500	10-12	2-4
Polymer						
Polypropylene						
Single filament	>0.16	As desired	0.91	80-110	0.5-0.7	18-25
Fibrillated strand	>0.16	As desired	0.91	55-60	1.0-1.2	6-10
Nylon	>0.16	As desired	1.1-1.4	60-130	0.35-0.60	13-25
Polyester	>0.16	As desired	1.4	105-127	1.4-1.6	10-11
PRD-49 (Kelvar)	5-7	As desired	1.4	400-430	12-19	2-4
Asbestos						
Chrysotile (white)	0.0008-0.8	4	2.6	72-500	12-28	3
Crocidolite (blue)	0.004-0.8	4	3.4			
Carbon	0.3	As desired	1.7-2.0	200-450	35-65	0.4-1.0
Vegetable						
Cotton	0.4-0.8	2-1/2	1.35	40-120	0.8-1.6	5-10
Sisal	0.3-1.9	48	1.48	120	--	3
Hemp	0.6-2.0	72	1.48	55	--	2
Glass & Ceramic Wools						
Rock (Scandanavian	0.08-0.2	2	2.7	70-110	10-17	~0.6
Alumino-Silicate	0.11	10	2.7	--	--	--

Table 9.2

Bond Improvement Techniques for Steel Fibers

Chemical Treatments	Bond improvement Factor	Mechanical Treatments	Bond Improvement Factor
Hot-dip galvanized	7.6	Looped end	>10.8†
Electrogalvanized (28-day)	6.3	Imitation Duoform*	>10.4†
Hot-dip galvanized + furnace	6.1	Solder blob (28-day)	> 9.3†
Hot-dip galvanized (28-day)	6.0	Duoform*	> 9.2†
Rusted (28-day)	5.8	Duoform* (28-day)	> 8.8†
Epoxide resin on wire + oven	5.7	Flat end	8.8
Electrogalvanized	5.2	Solder blob	6.5
Etched in phosphate (28-day)	5.2	Welded	5.9
Oxidized in furnace at 600°C	4.75	Curved end	5.6
Electrogalvanized + chromate	4.2	Zig-zag	4.7
Hot-dip galvanized + chormate	3.8	Spring-wound	4.2
Oxidized in furnace at 600°C (28-day)	3.7	Notched	3.8
Epoxide resin	3.1	Crushed	3.4
As received (28-day)	2.8	Abraded perpendicular	1.8
Cleaned (28-day)	2.3	Twisted	1.6
Oxidized in potassium dichromate	2.1	Abraded parallel	1.4
Vinyl resin on wire	2.05	NONE - AS RECEIVED	1.0
Rusted	2.0		
Oxided in furnace at 350°C	2.0		
Etched in concentrated nitric acid	1.6		
Copper-coated in $CuSO_4$	1.55		
Etched in dilute nitric acid	1.5		
Cleaned in solvent	1.5		
NONE - AS RECEIVED	1.0		

* Trademark of National-Standard Company, patented shape.

** Reference 242, data from G. H. Tattersall and C. R. Urbanawicz. (University of Sheffield), p. 17.

Table 9.3

Environmental Durability of Typical Fibers[162]

Fiber Materials	Resistance to Water	Resistance to Acids: Percent Loss in 25% HCl	Resistance to Alkalis: Percent Loss in 25% NaOH
Chrystalline silicates			
Chrysotile (white) asbestos	no significant attack	55.5% (refluxed for two hours) 56% (exposed at 26C for 528 hours)	1.0% (refluxed for for two hours) 1.0% (exposed at 26C for 528 hours)
Crocidolite (blue) asbestos	no significant attack	4.5% (refluxed for two hours) 3.0% (exposed at 26C for 528 hours)	1.5% (refluxed for two hours) 1.2% (exposed at 26C for 528 hours)
Glasses			
"E" glass	good	good	poor
"AR" glass	good	--	good
Ceramics			
Alumino-silicate	fair	fair	probably poor
Carbon	good	good	good
Metals			
Steel (high and low carbom)	poor except in alkali environment	poor	good
Stainless steel	good	good (to some acids)	good
Natural vegetables			
Cotton	poor	poor	fair
Sisal	poor	poor	--
Hemp	poor	poor	--
Polymers			
Polypropylene (filament)	good	good	good
Polypropylene (fibrillated)	fair	good	good
Nylon	good	fair (attacked by some)	good
Polyester	--	good	fair to poor

Table 9.4

Freeze-Thaw Durability Data*

Fiber Type	Air Entrainment	No. of Freeze & Thaw Cycles	Durability Factor, DFE
None	No	58	52
	Yes	300	94
Steel No. 1	No	24	42
	Yes	300	87
Steel No. 2	No	29	38
	Yes	300	91
Steel No. 3 (Stainless)	Yes	300	96
AR Glass	No	58	52
	Yes	300	78

* Unpublished WES report.

Table 9.5

Typical Impact Strengths for Mixed Fiber Reinforced Cement Composites[617]

Polymeric Fiber				Glass Fiber		Impact Strength, ft-lb/ft^2			
						Age, Days			
Type of Fiber	Denier	Length, in.	Fiber Volume, % Weight	Volume, % Weight	Curing* Conditions	7	28	180	365
Nylon	4	0.47	0.5	4.0	A	--	--	410	390
					W	910	--	385	420
					NW	--	--	385	390
					140F	--	390	--	--
Nylon	50	2.0	0.2	4.0	A	--	1315	--	--
					W	1370	1035	--	--
					140F	--	225	--	--
Nylon	50	2.0	0.9	4.5	A	--	2015	--	--
					W	2110	1705	--	--
					140F	--	770	--	--
Polypropylene	170 (Monofilament)	0.75	1.0	5.0	W	1705	1611	1345	1250
					NW	--	--	1970	1550
					140F	--	670	816	740
Polypropylene	170 (Monofilament)	2.0	1.2	4.3	A	--	2270	2140	2000
					W	2270	2235	1370	1370
					NW	--	2255	1945	1585
					140F	--	1165	1055	900
Polypropylene	170 (Monofilament)	2.0	2.3	4.0	A	--	3145	3050	3455
					W	4025	3500	2590	2290
					NW	--	3480	2555	2520
					140F	--	2205	2270	1715

(Continued)

Table 9.5 (Concluded)

Polymeric Fiber									
Type of Fiber	Denier	Length, in.	Fiber Volume, % Weight	Glass Fiber Volume, % Weight	Curing* Conditions	Impact Strength, ft-lb/ft^2 Age, Days 7	28	180	365
Polypropylene	6000 (Fiber)	0.47	1.0	5.0	W	1365	1140	1115	920
					NW	--	--	1455	1535
					140F	--	555	--	265
Polypropylene	1000 (Fiber)	0.47	1.0	5.0	W	1290	1000	900	960
					NW	--	--	1310	1075
					140F	--	560	--	425
Polypropylene	1000 (Fiber)	2.0	1.0	4.3	A	--	1920	1825	1725
					W	1840	1740	1010	815

*A = air cured; W = water cured; NW = natural weathering.

Table 9.6

Typical Impact Strengths for Nylon and Polypropylene Fiber Reinforced Cement Composites[617]

Denier	Length in.	Fiber Volume % Weight	Curing Conditions	Impact Strength, ft-lb/ft^2 Age, Days 7	28	60	180	365
Nylon (Monofilament)								
24	1	0.5	W	510	--	--	--	--
24	1	2.0	W	1495	--	--	--	--
			140F	--	--	1295	--	--
24	1	4.0	W	2010	--	--	--	--
			140F	--	--	2040	--	--
4	0.5	1.0	W	540	--	--	--	--
			140F	--	--	355	--	--
50	2	0.9	A	--	575	--	--	--
			W	925	660	--	--	--
			140F	--	750	--	--	--
Polypropylene (Fiber)								
1000	0.8	1.0	W	445	--	400	385	430
			NW	--	--	--	410	430
			140F	--	--	420	430	420
1000	2	1.0	W	555	--	--	--	--
			140F	--	--	495	495	--
12,000	0.8	2.0	W	625	--	--	--	--
			140F	--	--	335	575	--

(Continued)

Table 9.6 (Concluded)

Denier	Length in.	Fiber Volume % Weight	Curing Conditions*	Impact Strength, ft-lb/ft^2 Age, Days 7	28	60	180	365
Polypropylene (Monofilament)								
170	0.8	0.8	W	875	875	--	755	750
			NW	--	--	--	750	--
			140F	--	--	830	765	780
170	1	1.0	A	--	800	--	--	985
			W	800	755	--	--	890
			NW	--	--	--	--	865
			140F	--	800	--	--	--
170	2	1.0	A	--	1325	--	1250	1200
			W	1275	1420	--	1475	1225
			NW	--	--	--	1055	1200
			140F	--	1235	--	--	--
170	2	2.3	A	--	1790	--	1885	--
			W	2000	1860	--	1740	1695
			NW	--	--	--	--	1453
			140F	--	1905	--	1680	--
170	2	2.8	A	--	2605	--	2990	2290
			W	4060	2860	--	2410	2290
			NW	--	--	--	--	2305
			160F	--	2650	--	2630	--

*A = air cured; W = water cured; NW = natural weathering.

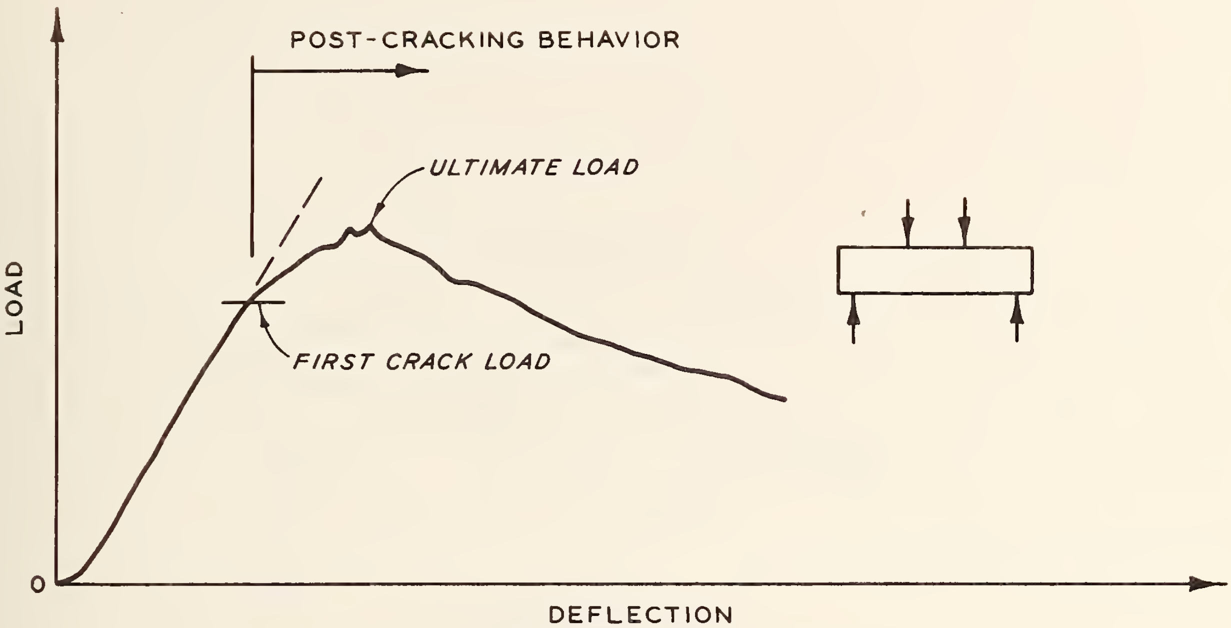


Figure 9.1. Typical flexural load-deflection behavior of steel fiber concrete

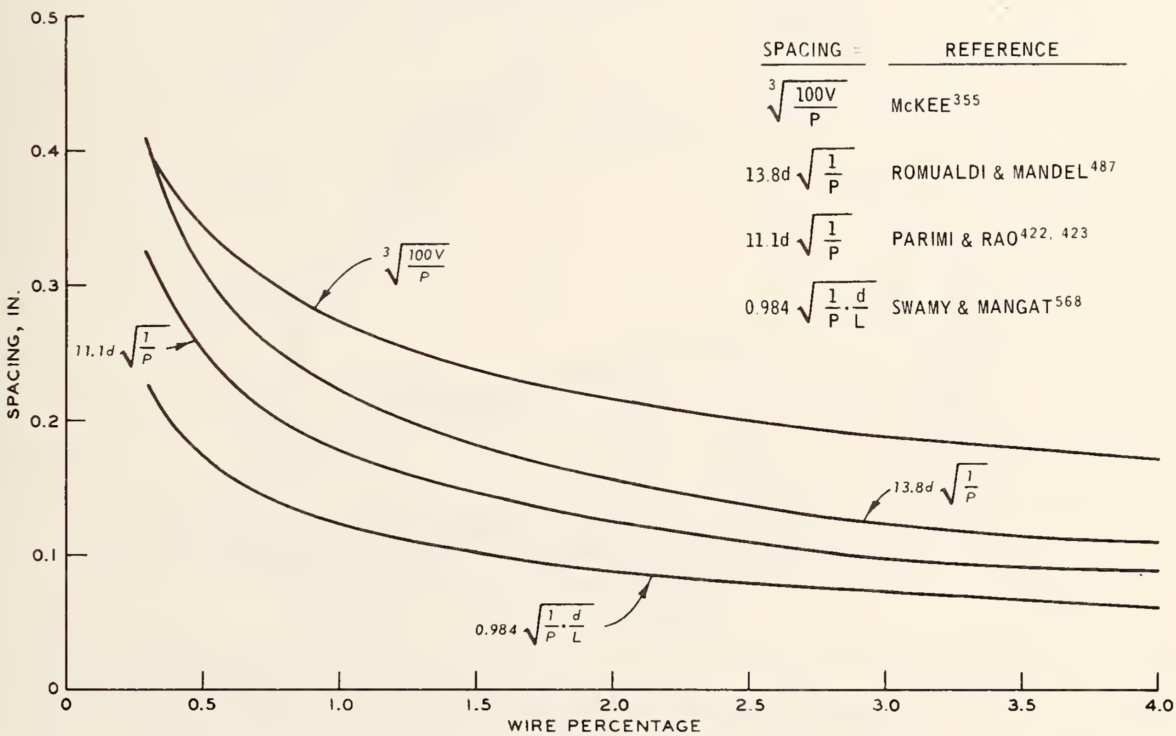


Figure 9.2. Fiber spacing relationships

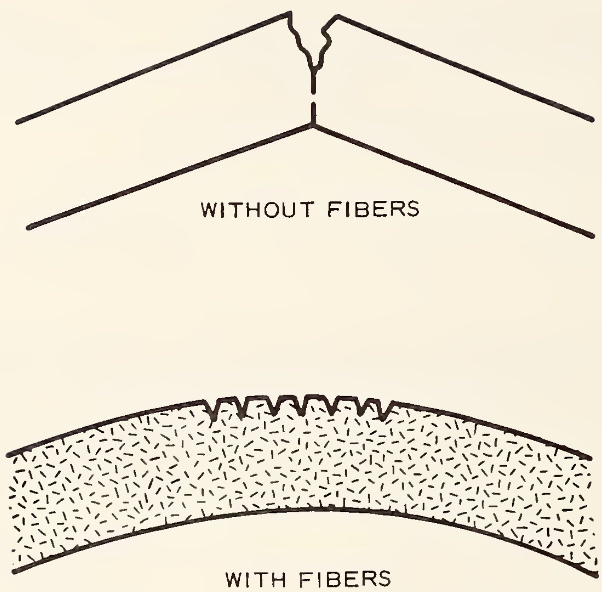


Figure 9.3. Idealized crack arrest mechanism in bending

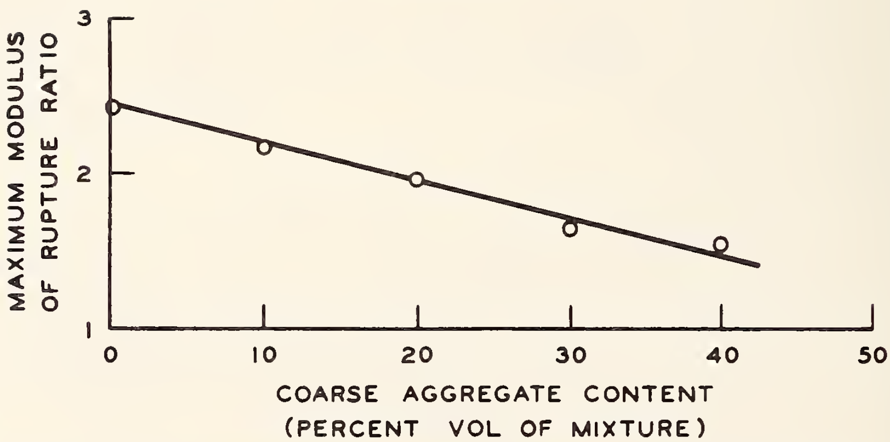


Figure 9.4. Effect of aggregate content on the flexural strength of steel fiber reinforced concrete[568]

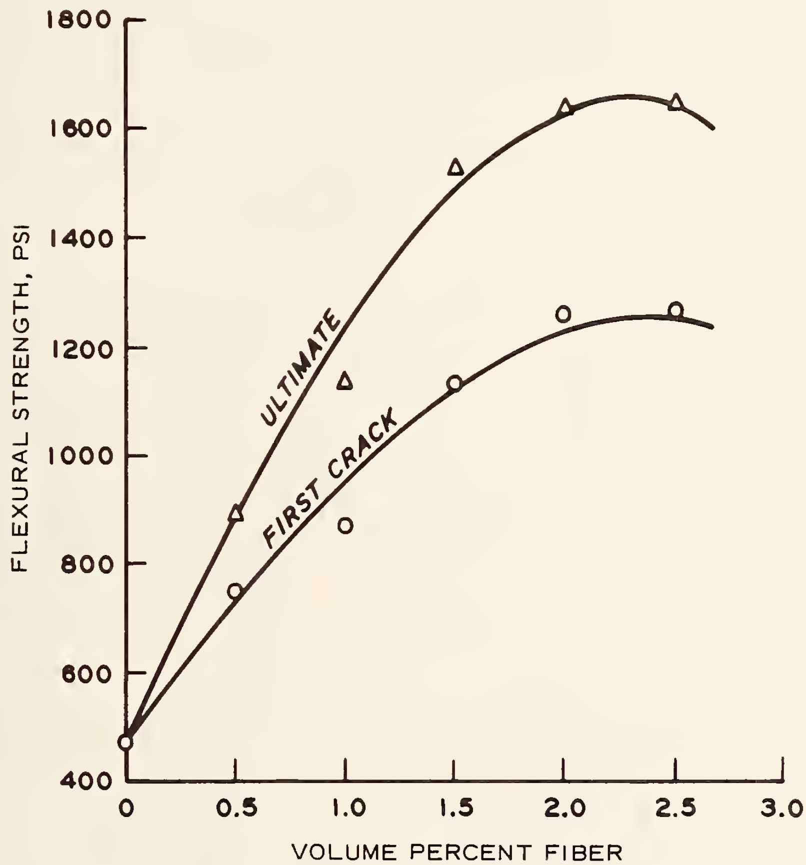
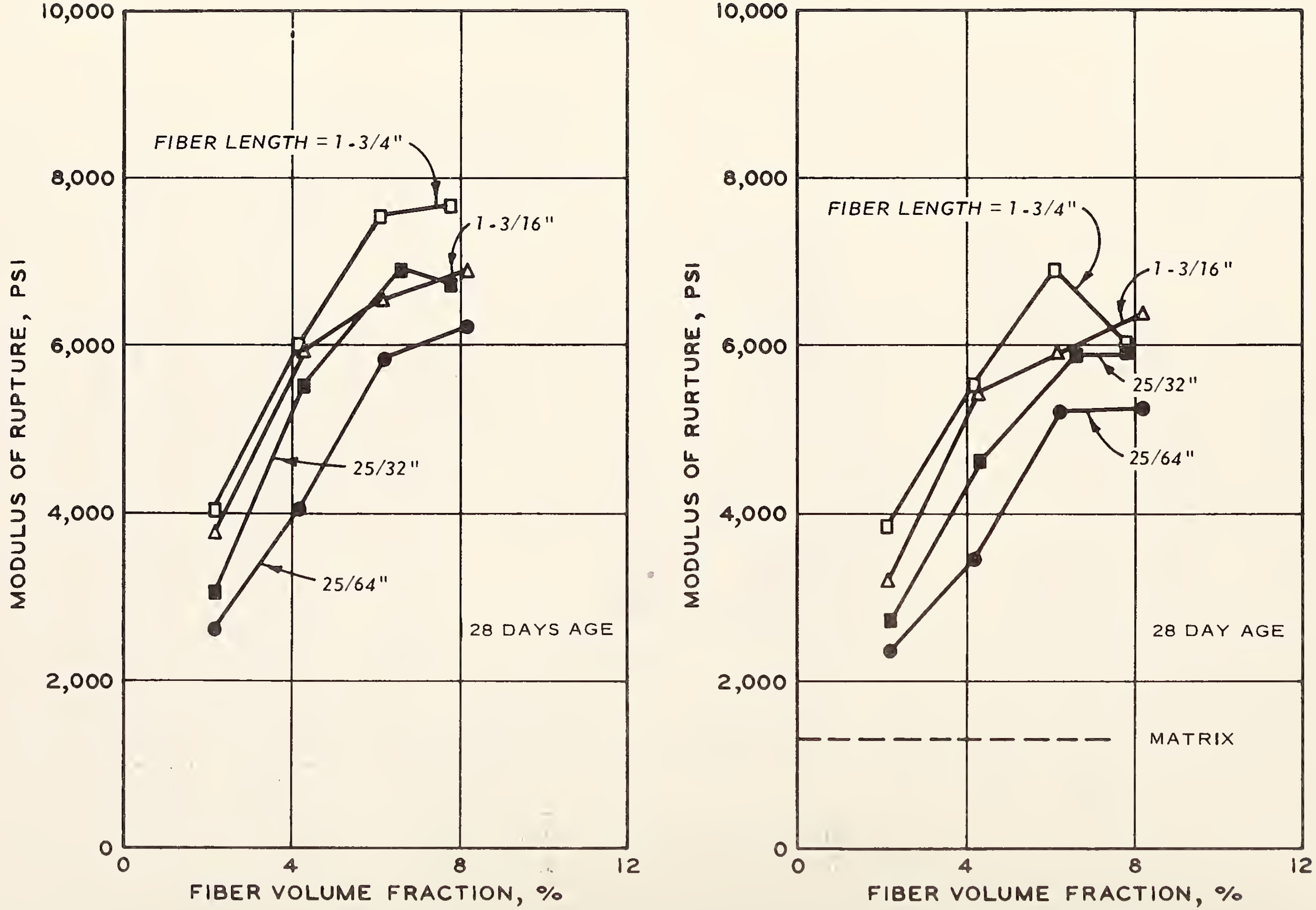


Figure 9.5. Effect of fiber content on flexural strength of glass fiber reinforced concrete[344]



a. AIR STORAGE

b. WATER STORAGE

Figure 9.6. Relation between fiber volume fraction and modulus of rupture of glass fiber reinforced cement composites[15]

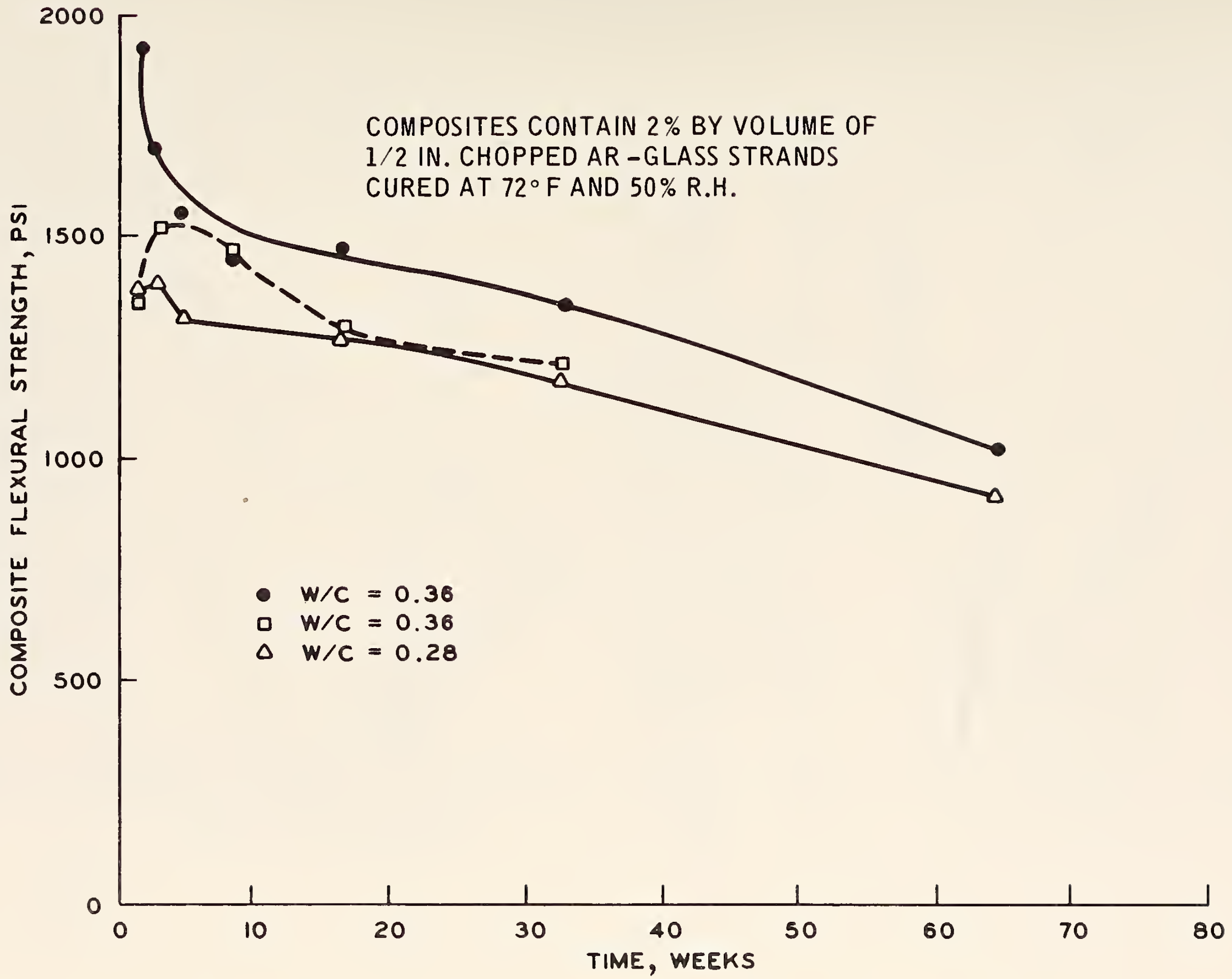


Figure 9.7. Flexural strength versus age relation for AR-glass reinforced cement composites[106]

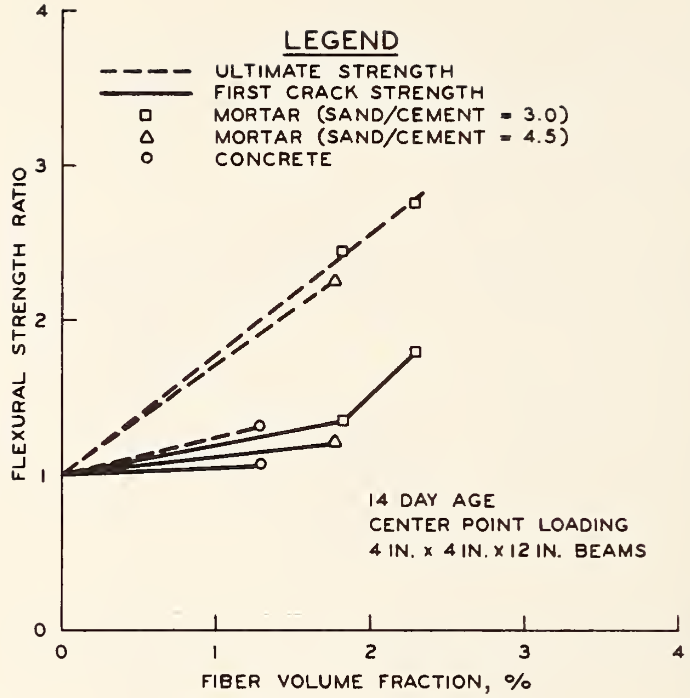


Figure 9.8. Flexural strength ratio versus fiber volume fraction for 2-3/4 in. fibrillated polypropylene fiber reinforced mortars and concrete[146]

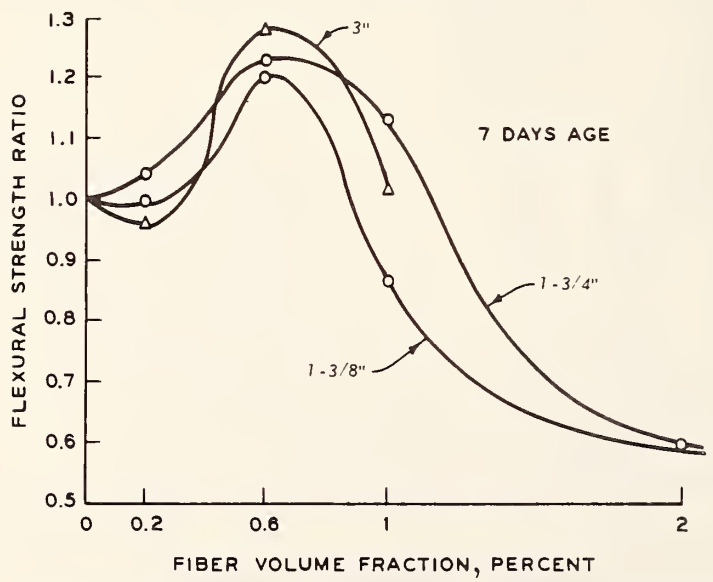


Figure 9.9. Flexural strength ratio versus fiber volume fraction for various lengths of fibrillated polypropylene fiber reinforced concrete[117]

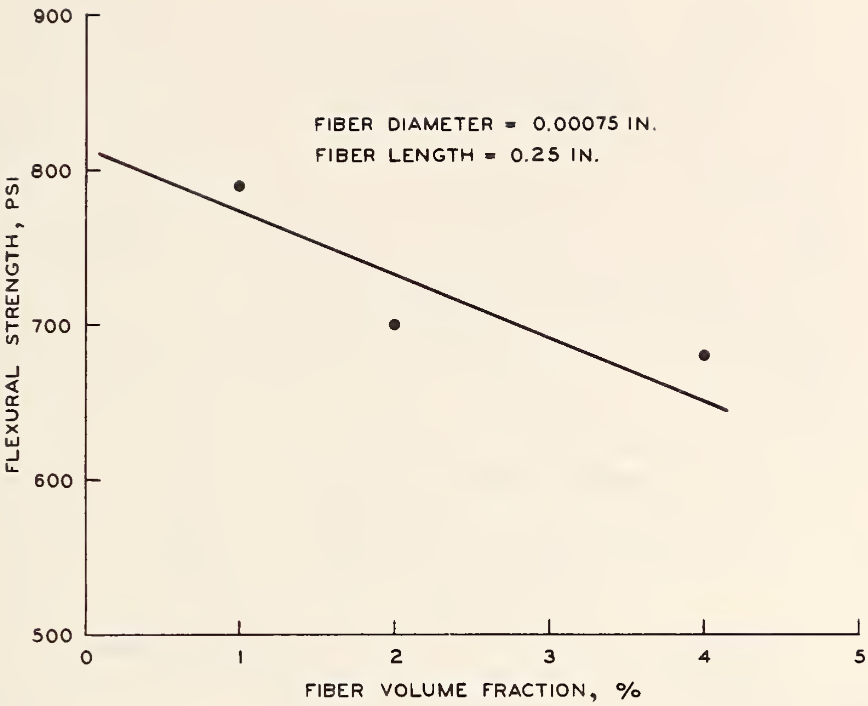


Figure 9.10. Flexural strength versus fiber volume fraction for nylon fiber reinforced mortar[367]

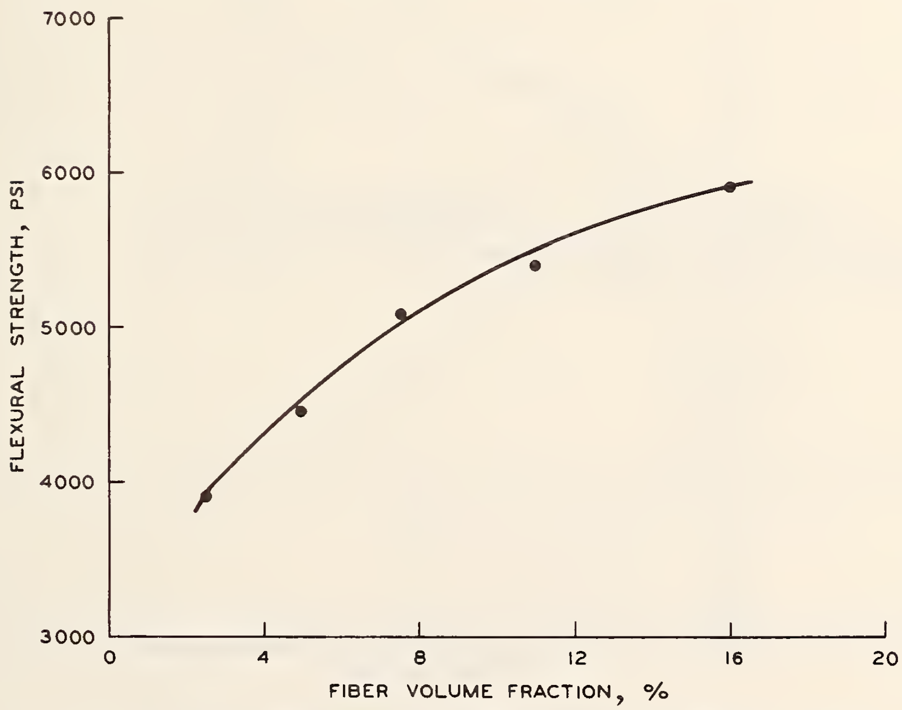


Figure 9.11. Flexural strength versus fiber volume fraction for asbestos fiber reinforced concrete[287]

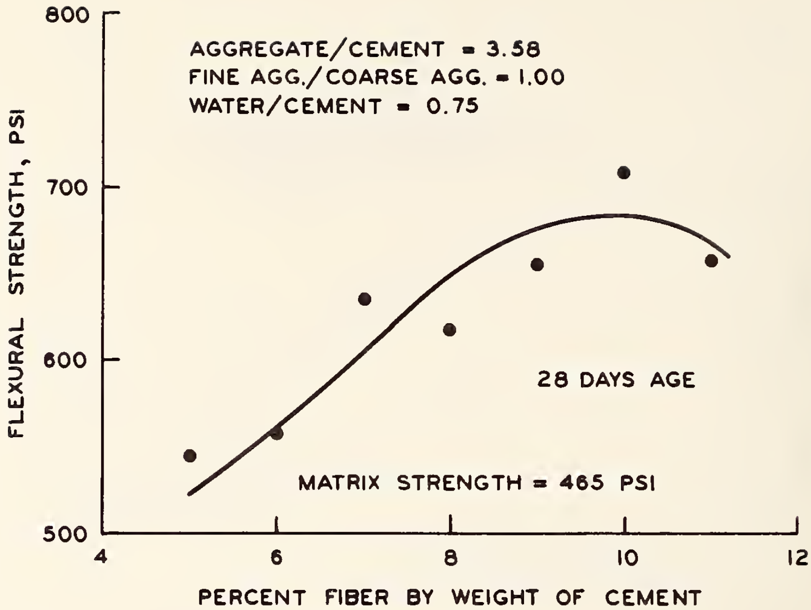


Figure 9.12. Flexural strength versus fiber content relation for asbestos fiber reinforced concrete[242]

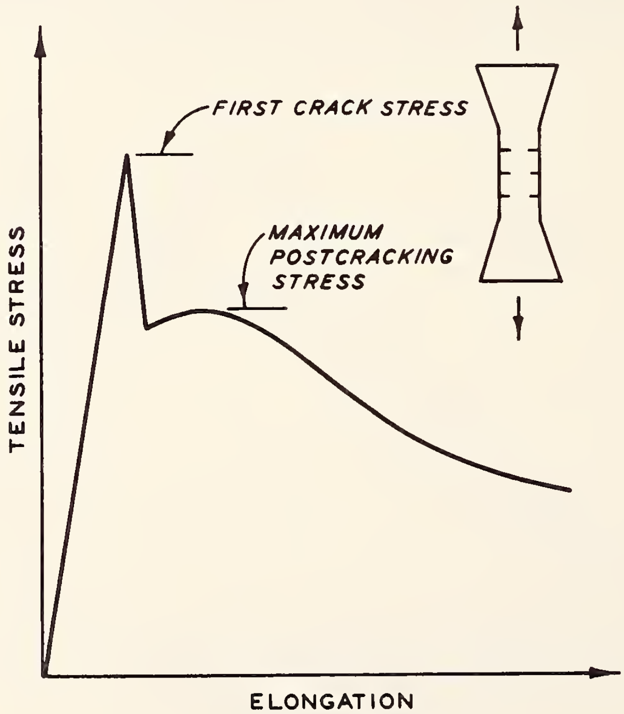


Figure 9.13. Idealized average tensile stress versus elongation relation for steel fiber reinforced mortar[377]

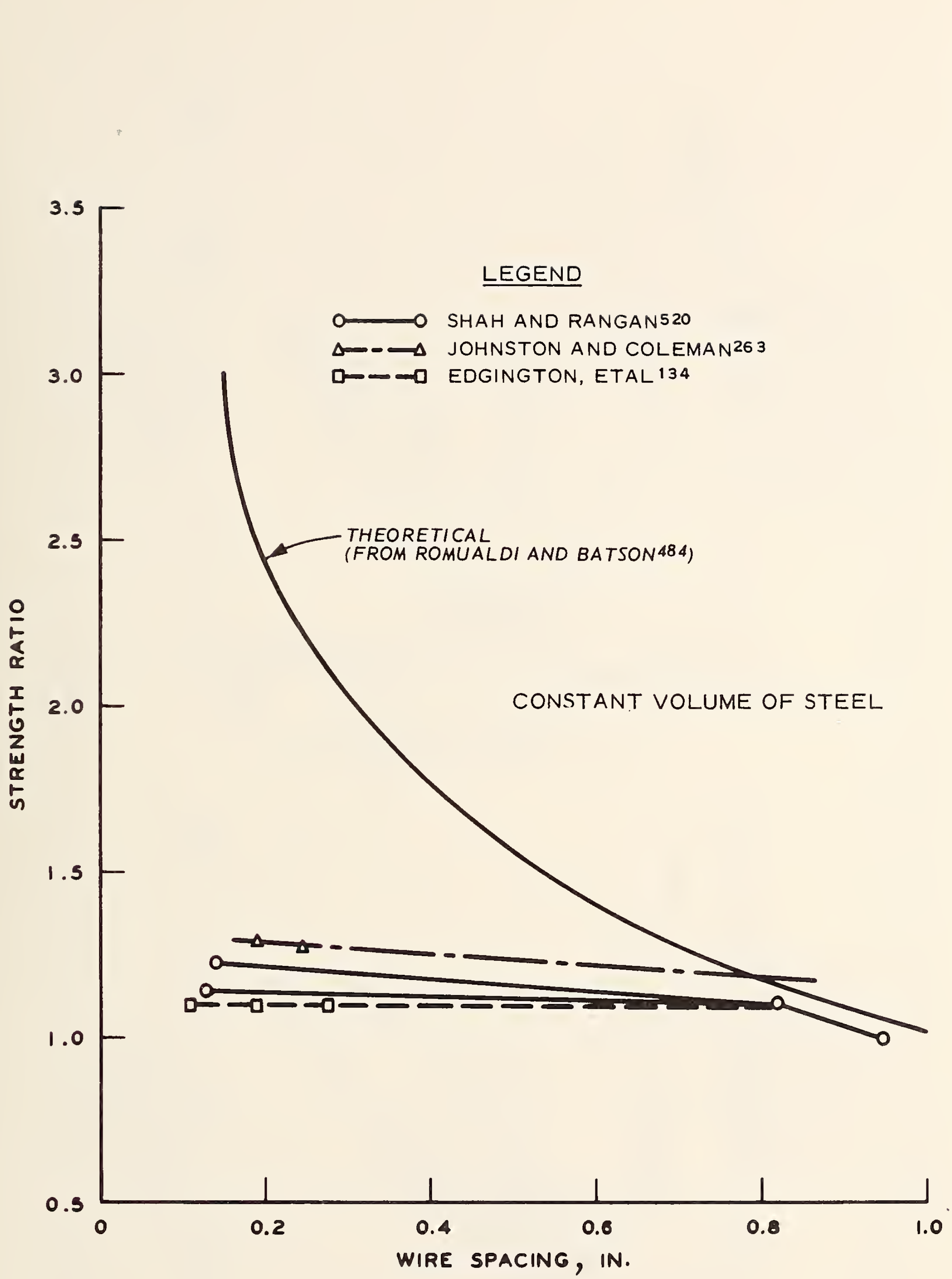
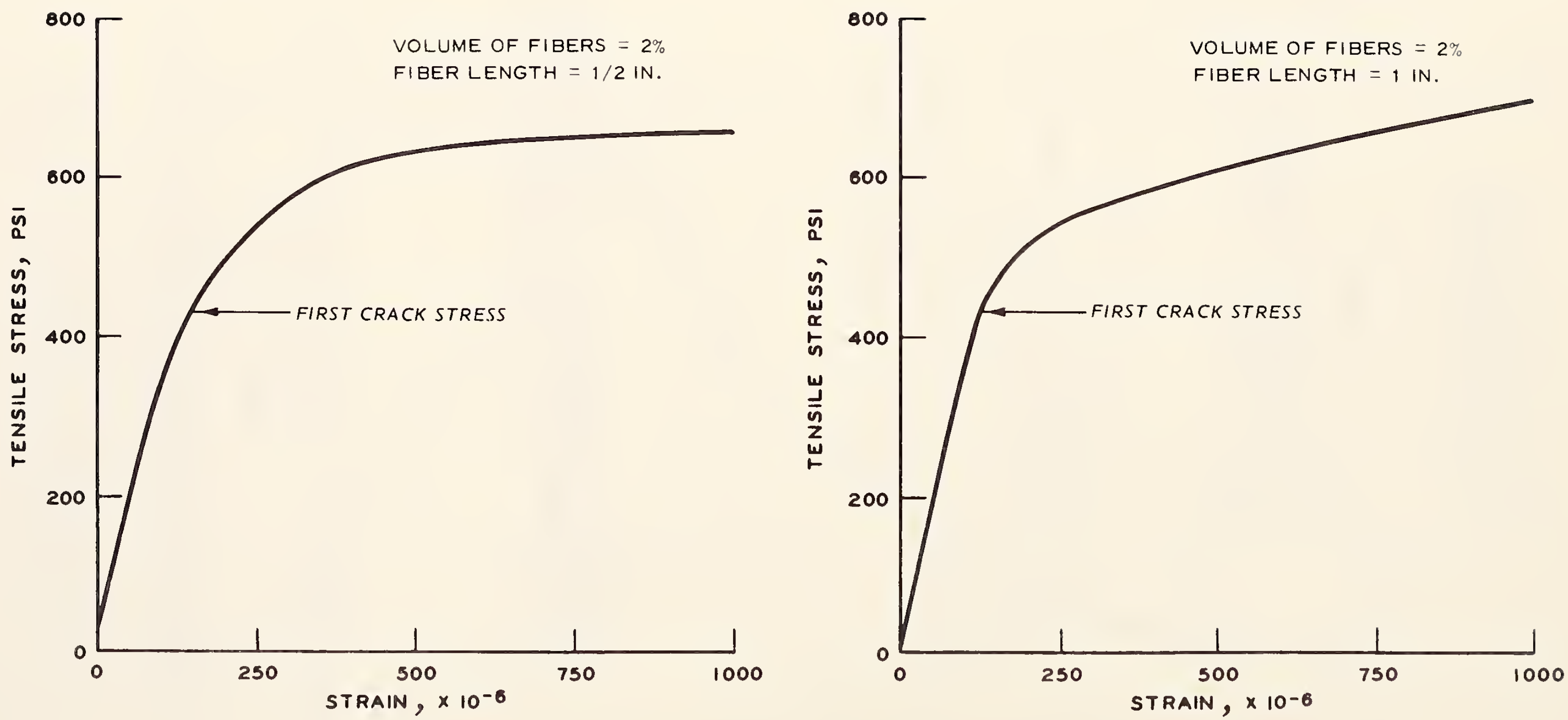


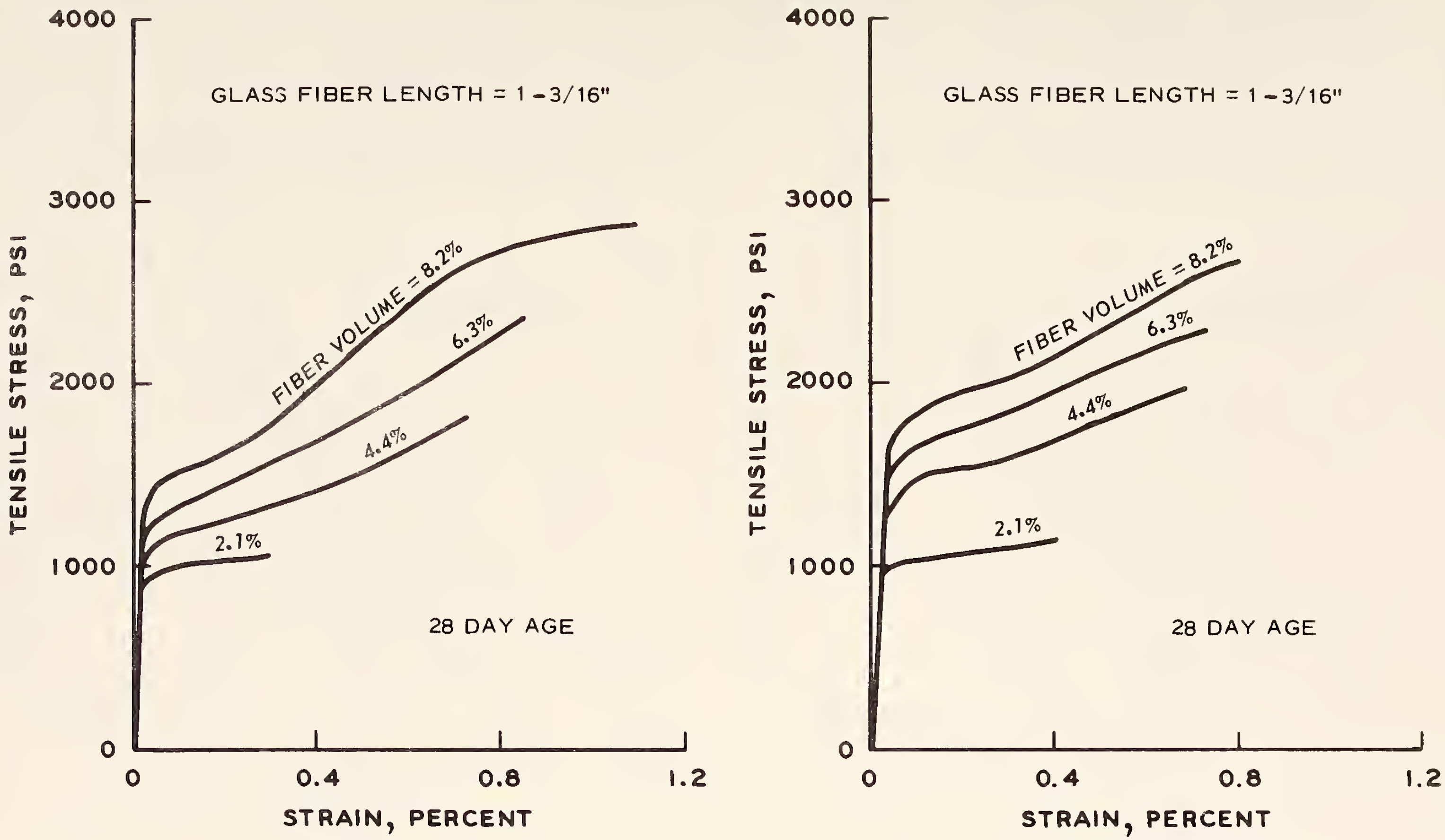
Figure 9.14. Effect of spacing of reinforcement on cracking strength of steel fiber reinforced concrete[134]



a. GLASS FIBERS

b. STEEL FIBERS

Figure 9.15. Typical stress-strain curves in tension of fiber reinforced mortar[519]



a. STORED IN AIR b. STORED IN WATER

Figure 9.16. Tensile stress-strain curves of glass fiber reinforced cement composites of varying fiber contents and curing conditions[15]

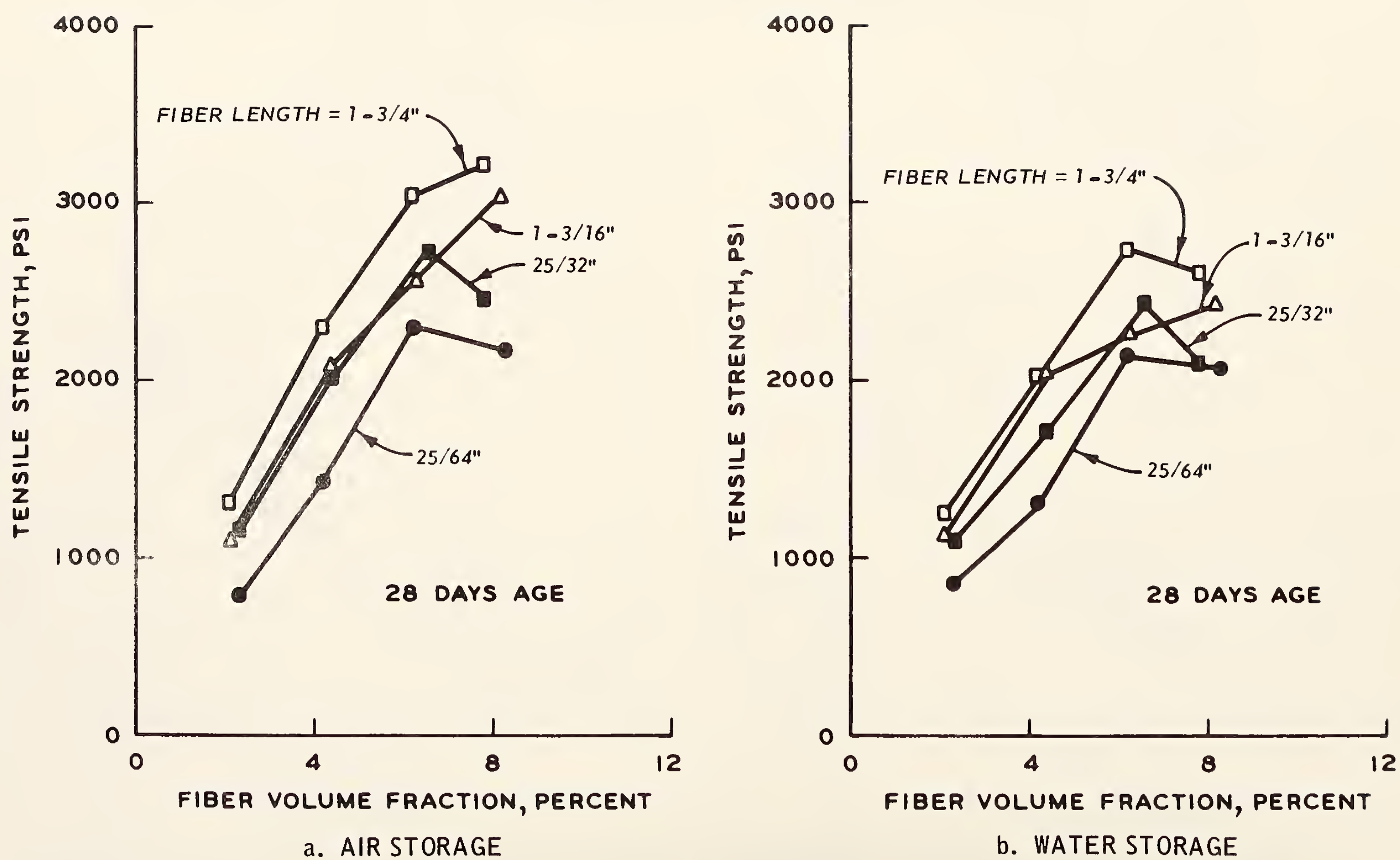


Figure 9.17. Relation between fiber volume fraction and tensile strength of glass fiber reinforced cement composites[15]

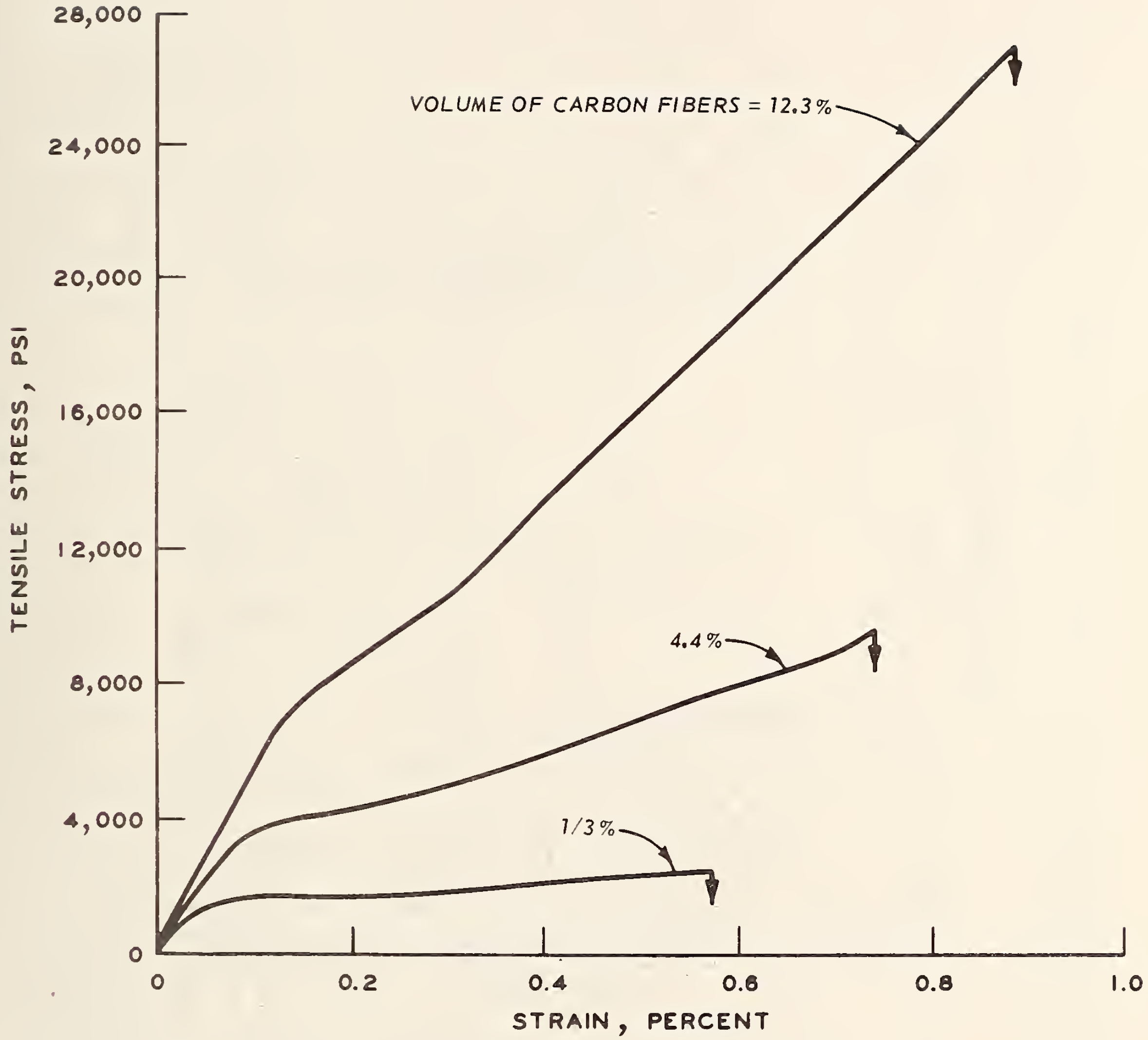


Figure 9.18. Tensile stress versus strain relations for continuous carbon fiber reinforced cement composites[33]

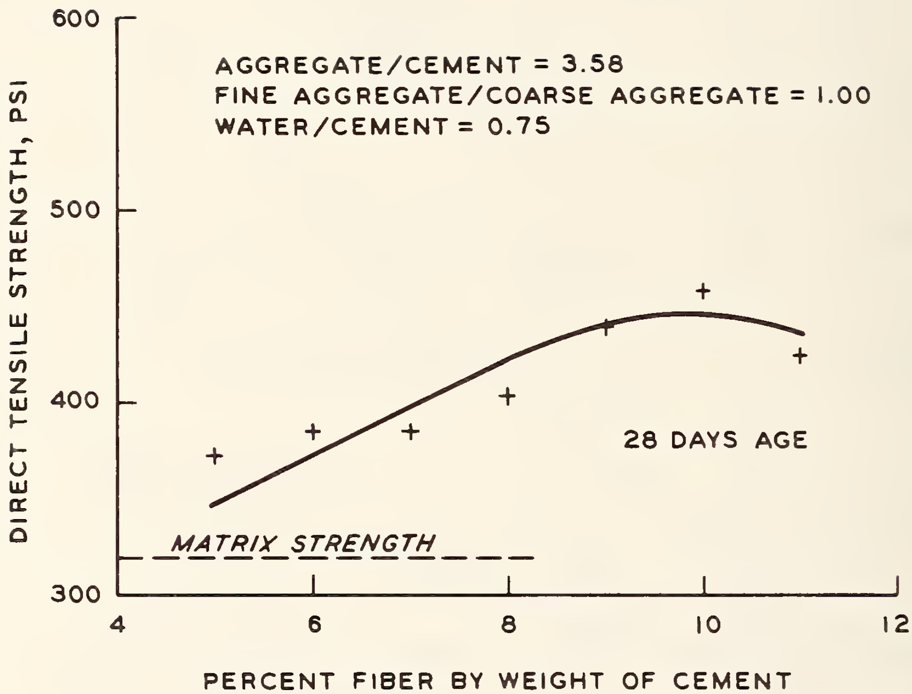


Figure 9.19. Tensile stress versus fiber content relation for asbestos fiber reinforced concrete[242]

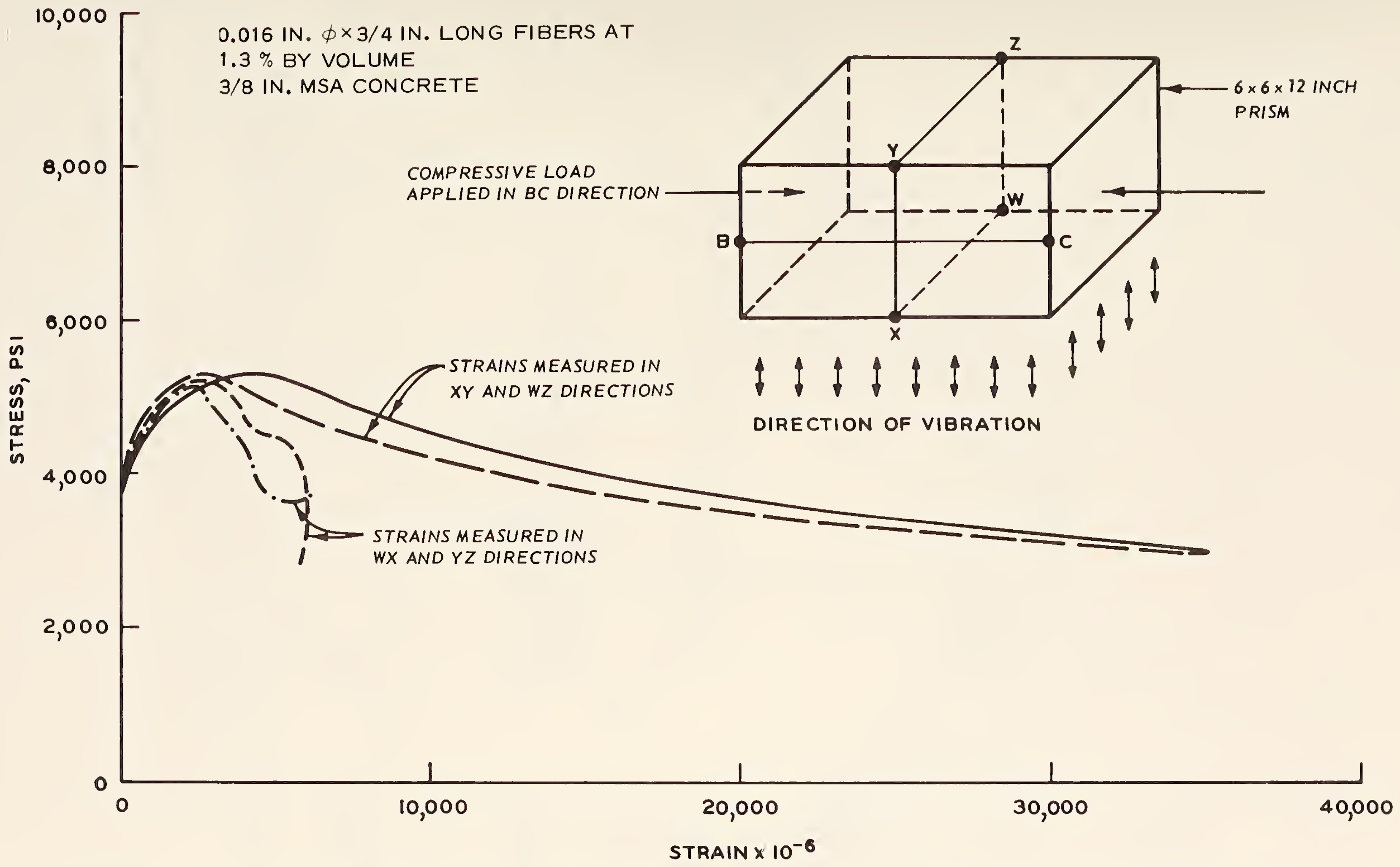


Figure 9.20. Compressive stress versus strain relations for steel fiber reinforced concrete showing the effects of fiber orientation due to external vibration[133]

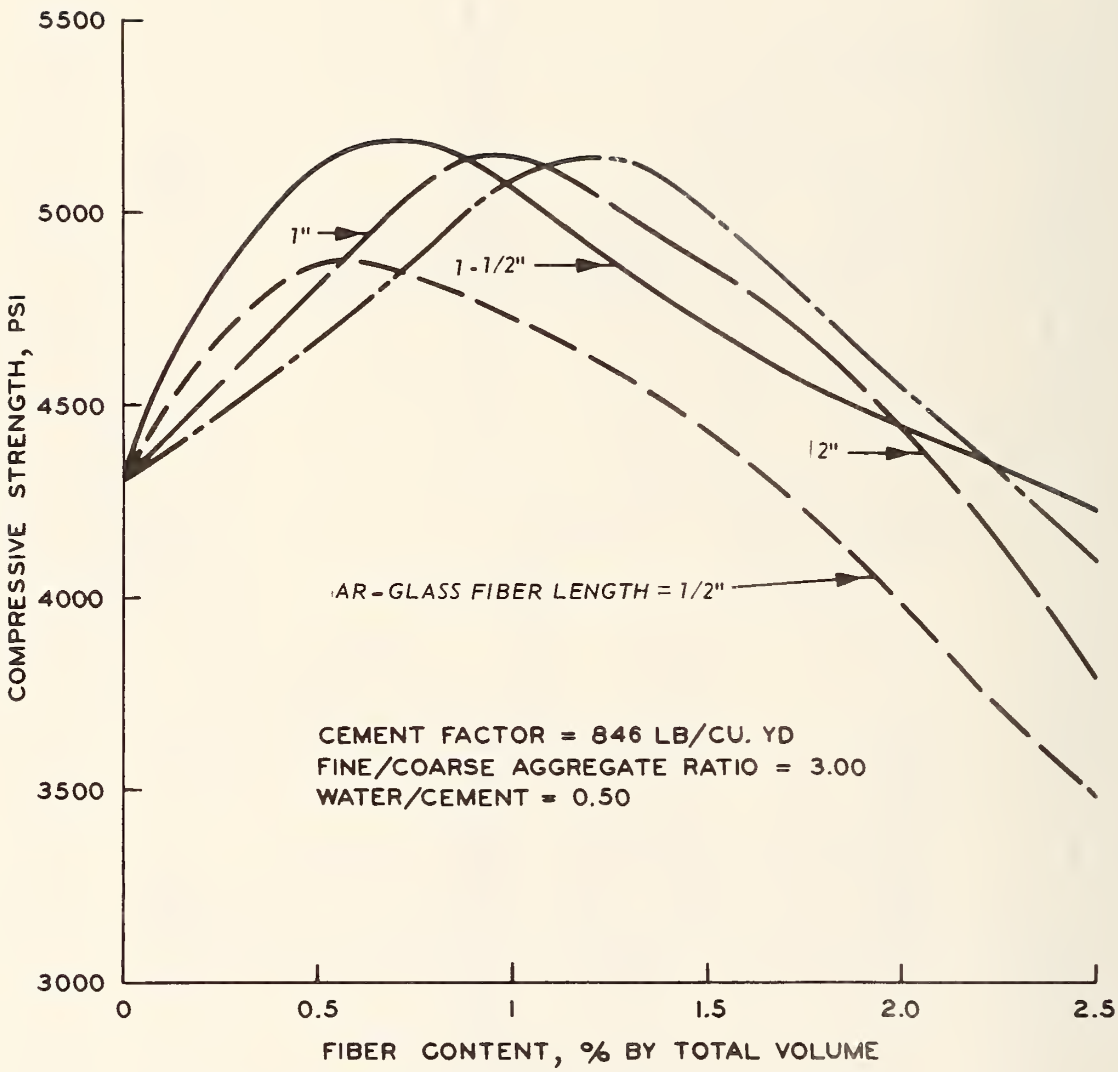


Figure 9.21. Compressive strength versus fiber content for AR glass fiber reinforced concrete[344]

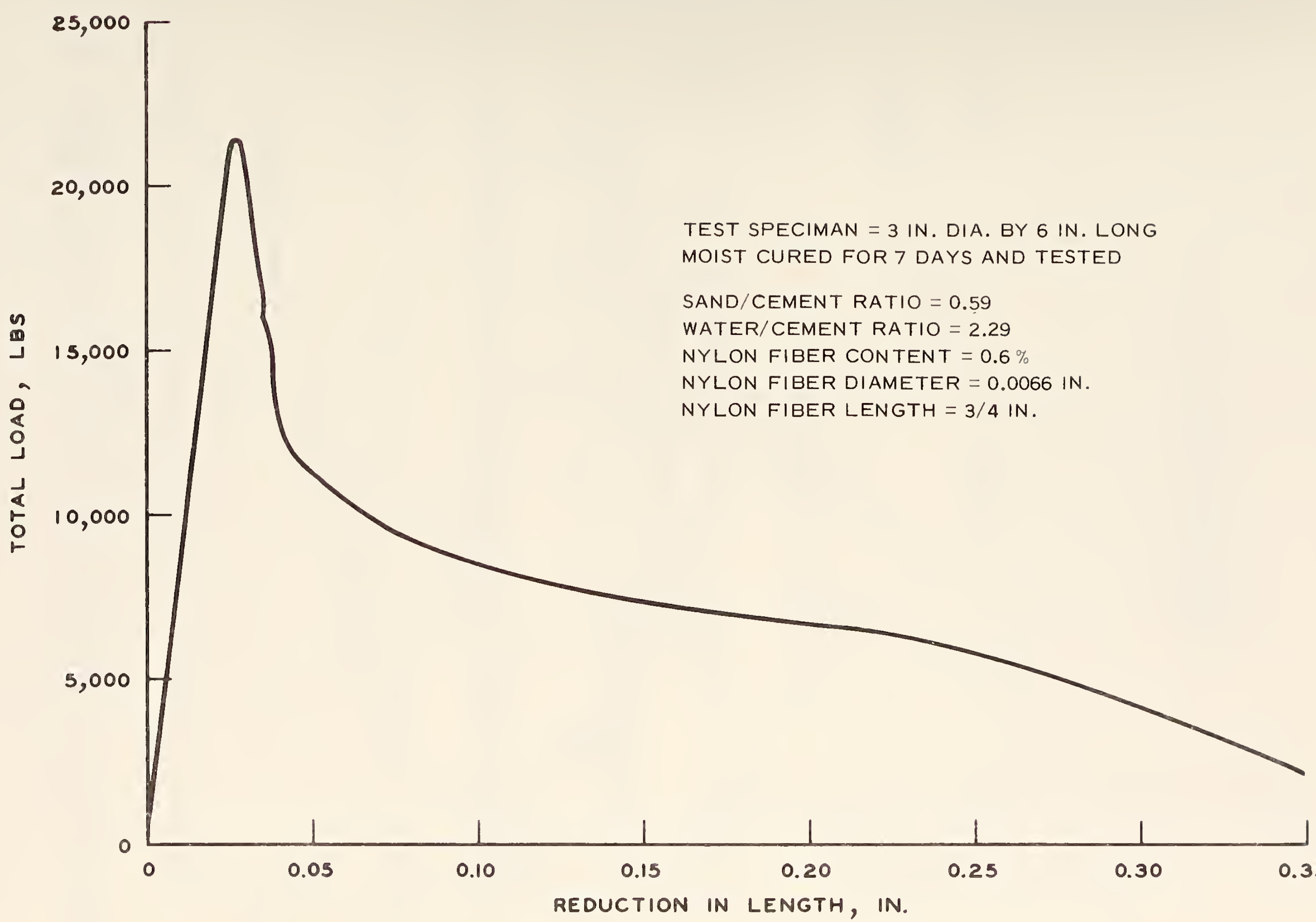


Figure 9.22. Total compressive load versus deformation curve for nylon fiber reinforced mortar[74]

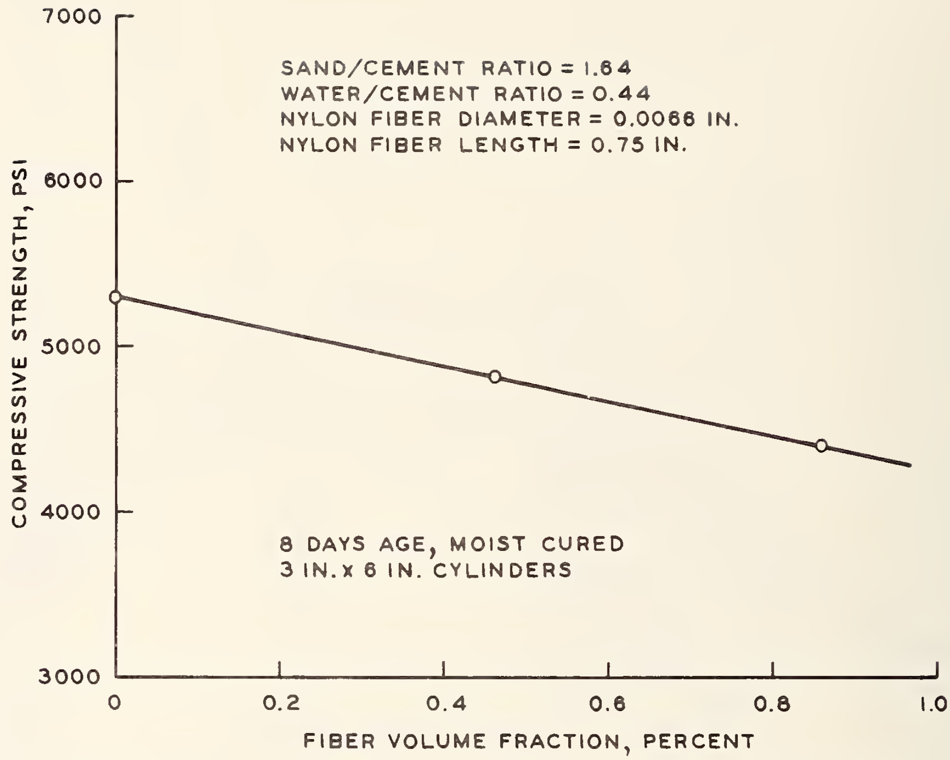


Figure 9.23. Compressive strength versus fiber volume fraction for nylon fiber reinforced mortar[74]

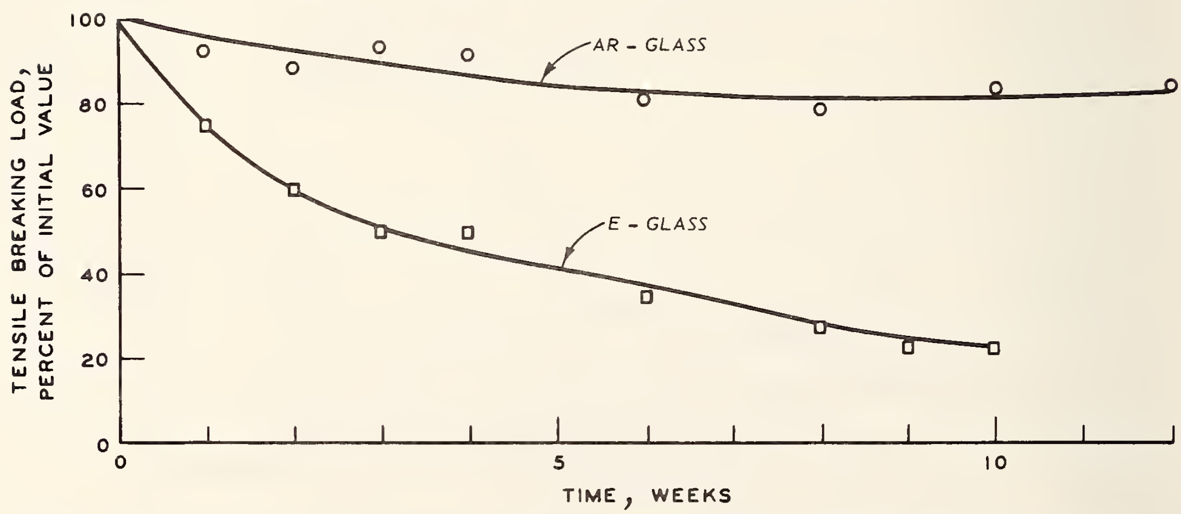


Figure 9.24. Reduction of tensile breaking load with time for glass strands exposed to cement effluent solution at 75F[106]

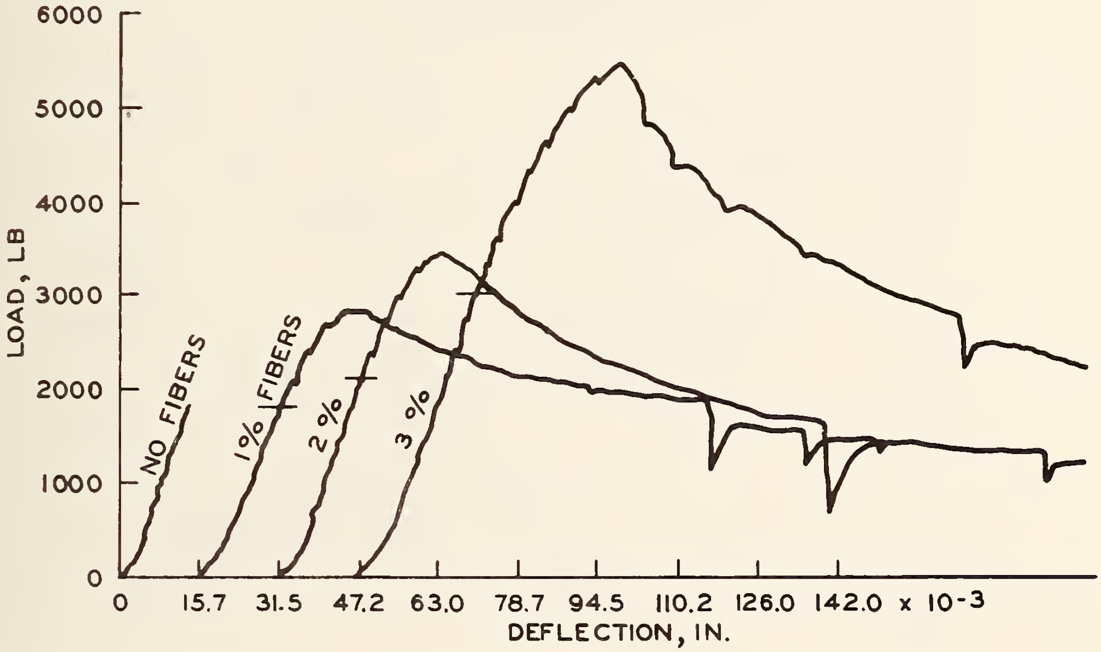


Figure 9.25. Typical load-deflection characteristics of steel fiber reinforced mortar[555]

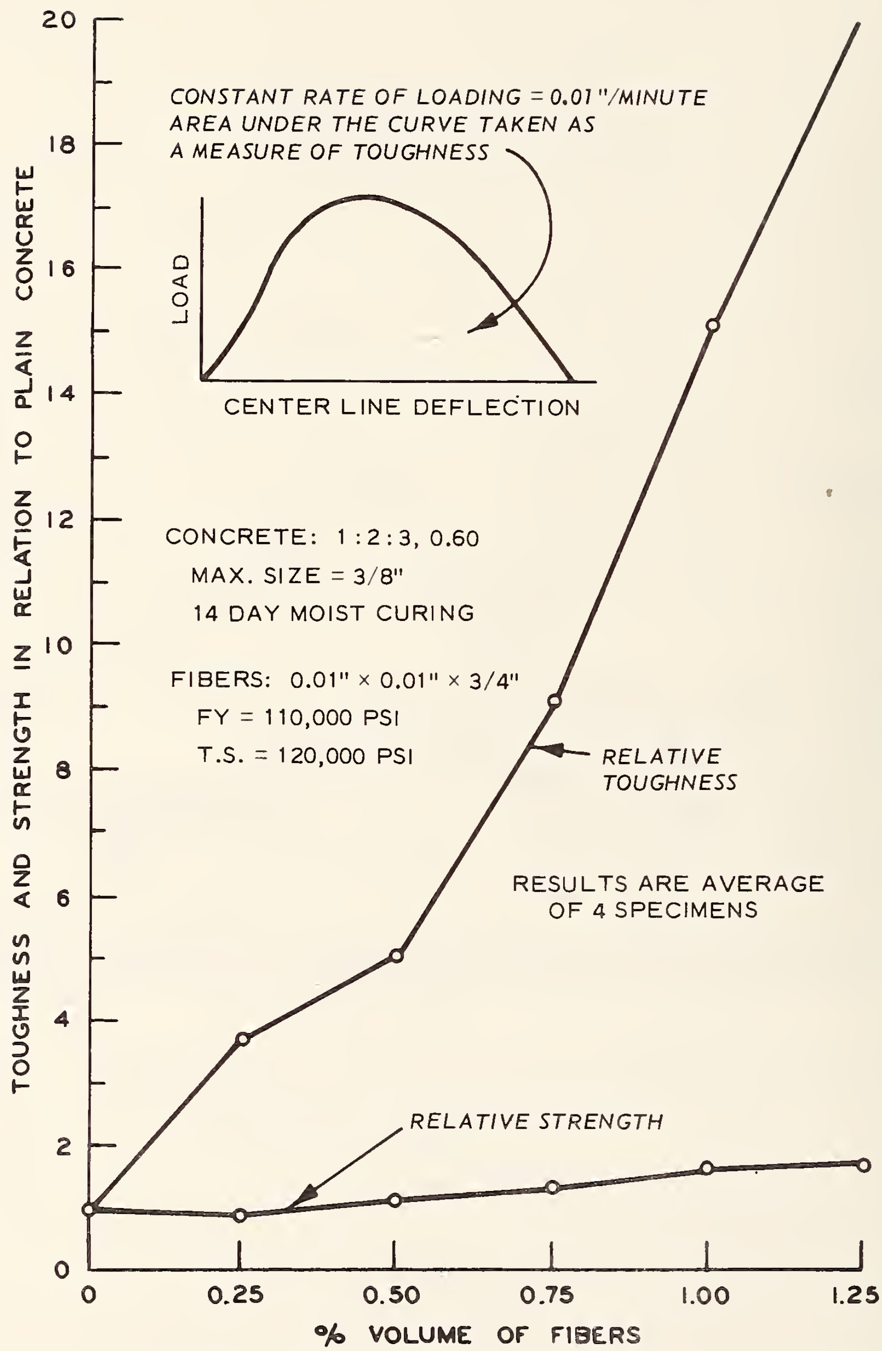


Figure 9.26. Effect of volume of fibers on flexural toughness and strength[521]

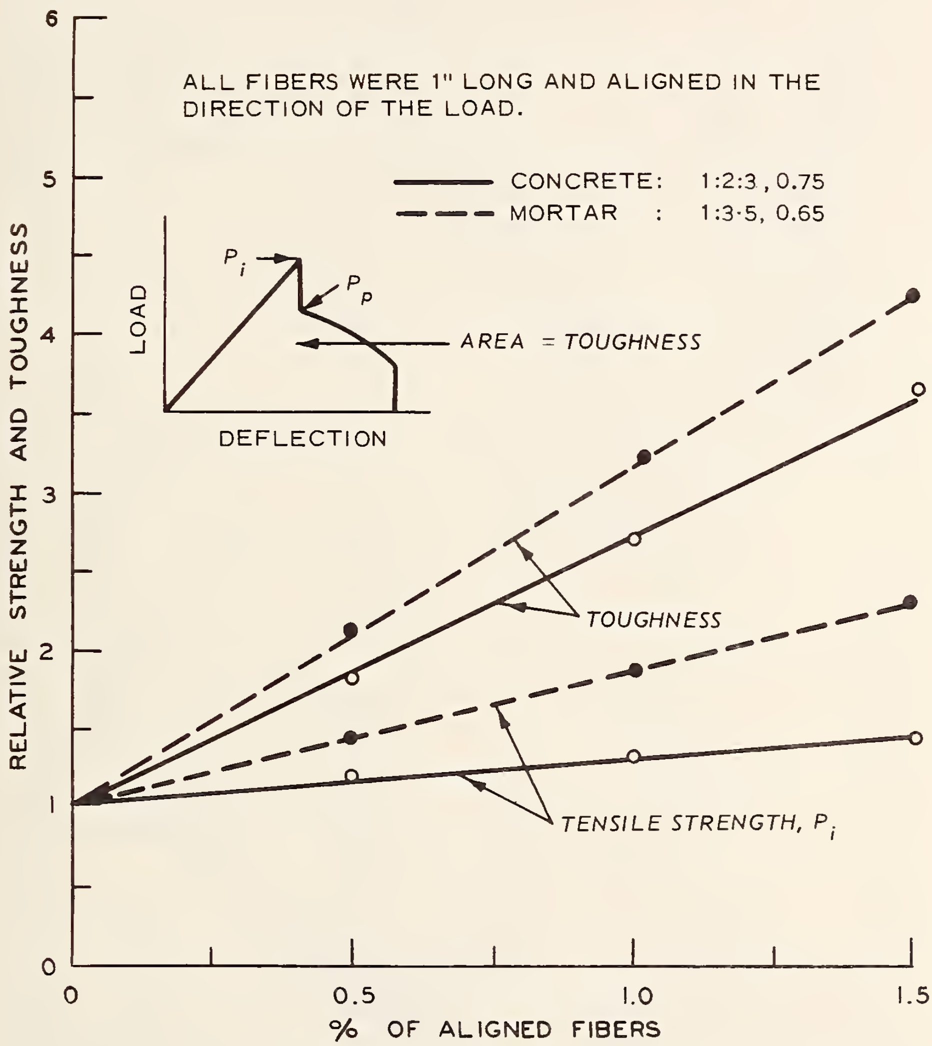


Figure 9.27. Effect of fiber alignment on tensile toughness and strength[521]

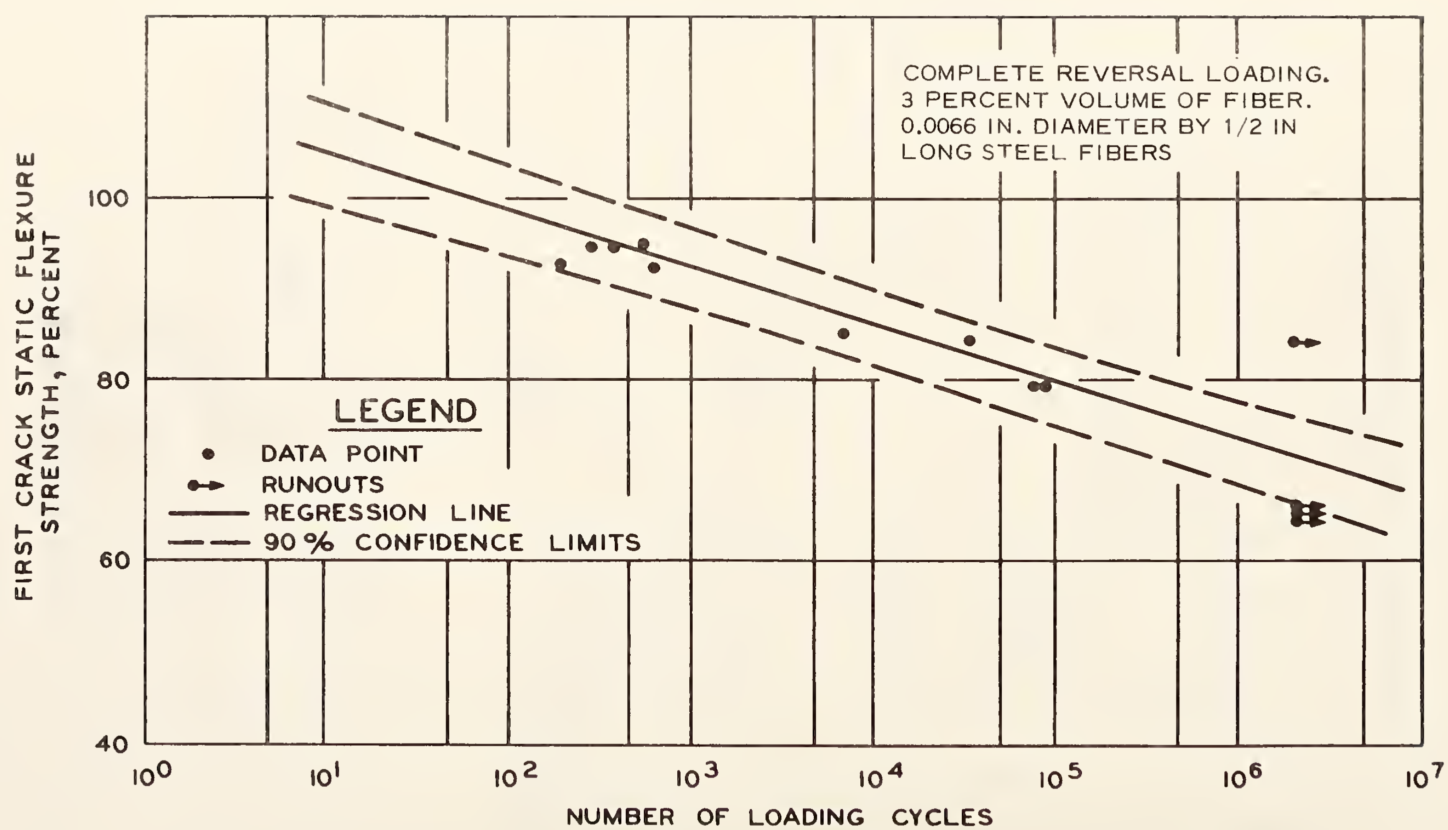
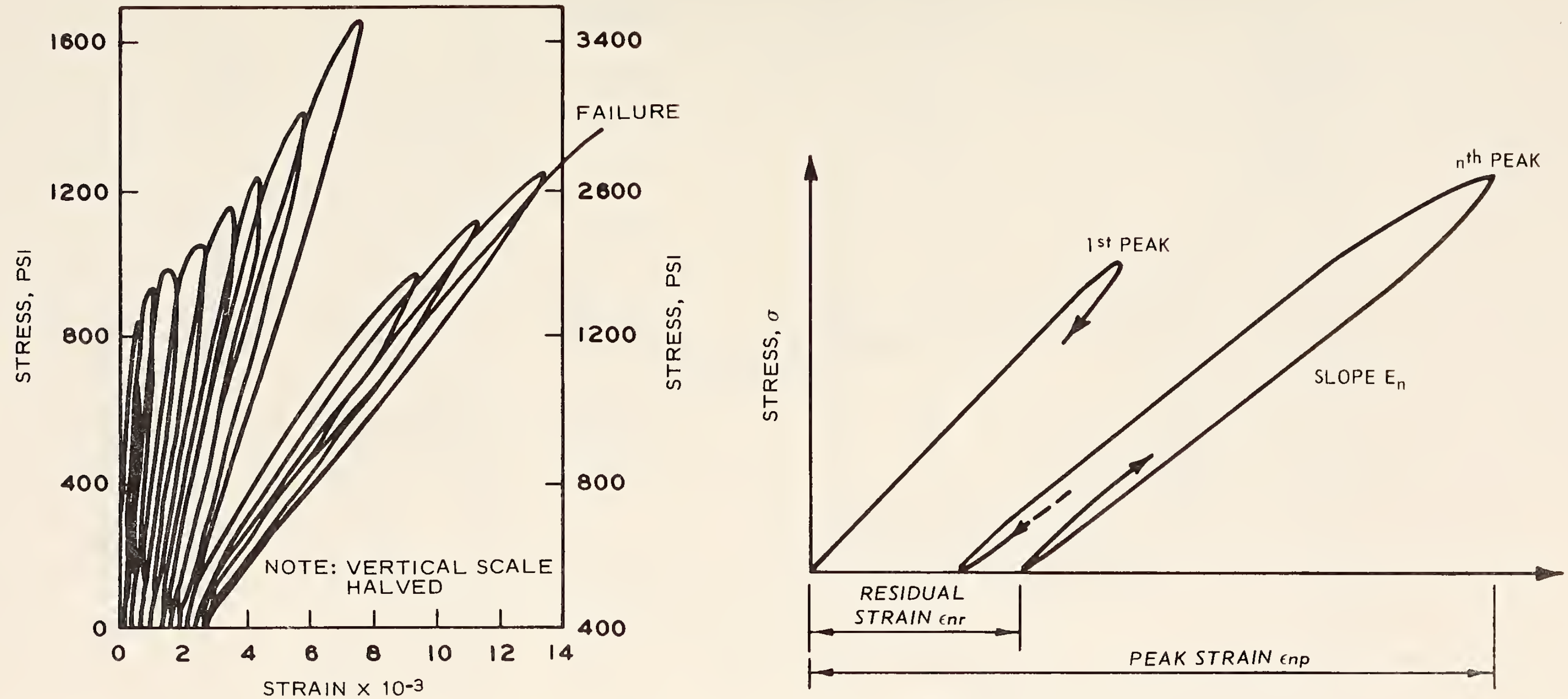


Figure 9.28. Fatigue strength for steel fiber reinforced concrete[50]



a. TEST RESULT

b. DESCRIPTION OF CYCLIC TENSILE STRESS-STRAIN CURVE

Figure 9.29. Tensile stress-strain curve for glass fiber cement under cyclic loading[20]

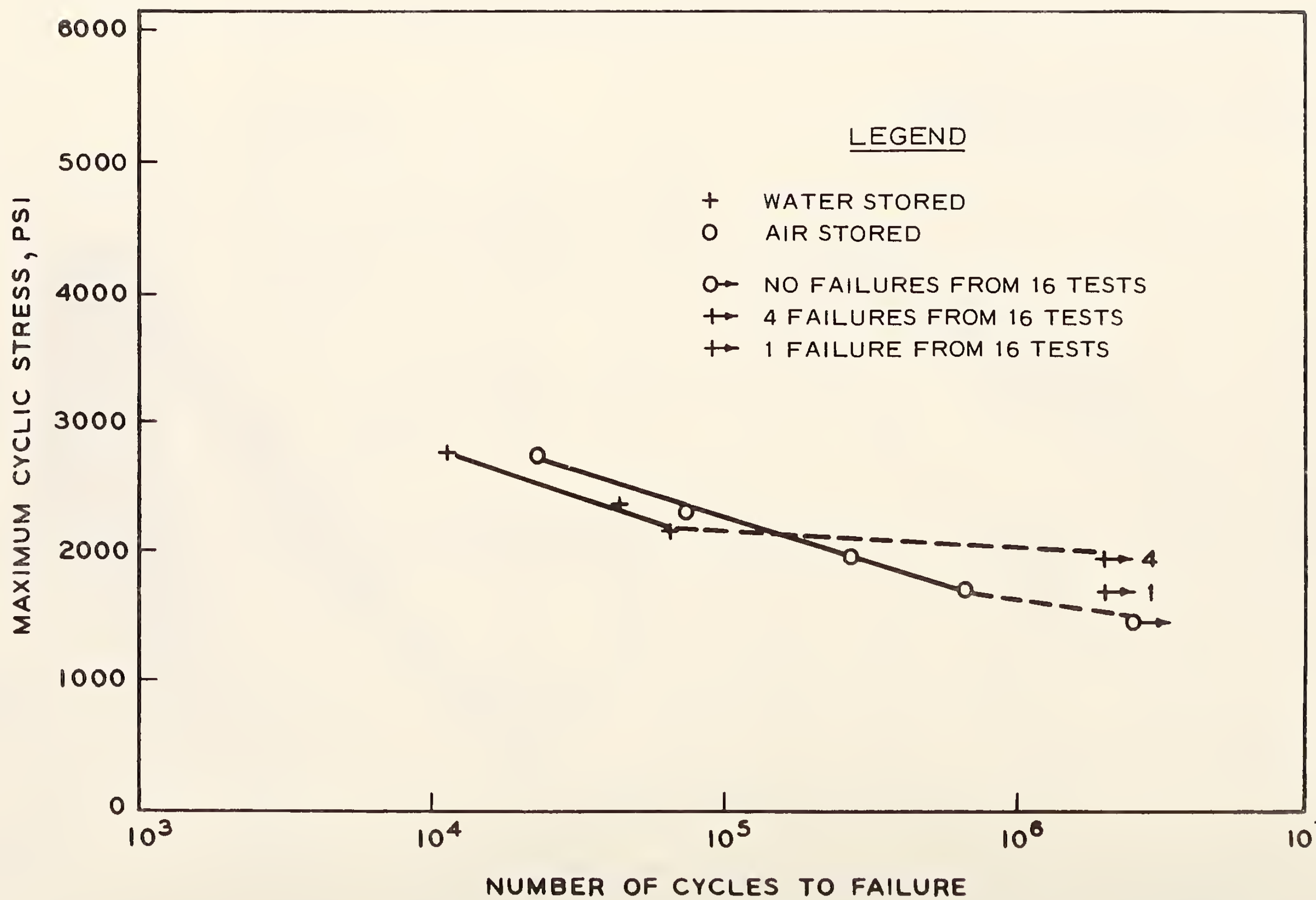
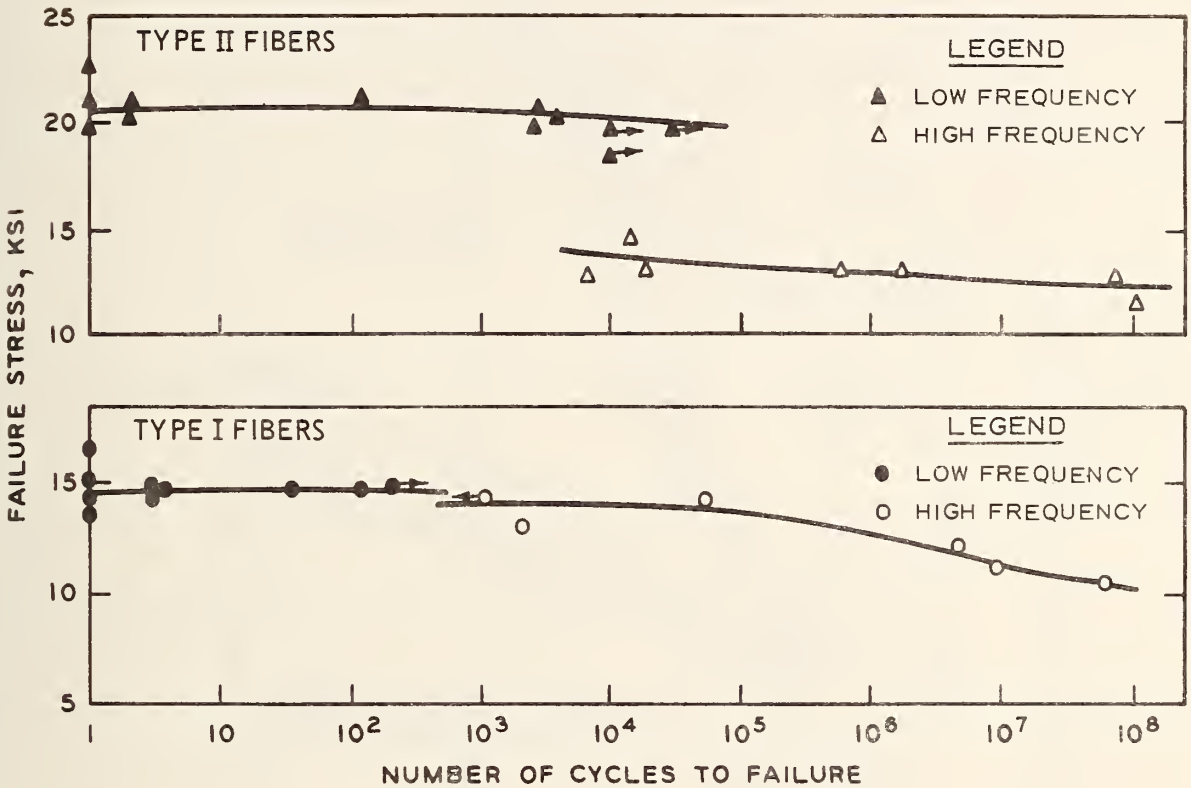


Figure 9.30. Fatigue life of glass fiber reinforced cement[288]



NOTE: ALL COMPOSITES HAVE 8% VOLUME OF FIBERS.
ARROWS POINTING RIGHT INDICATE SPECIMENS WHICH HAD NOT FAILED WHEN TEST STOPPED.
ARROWS POINTING LEFT INDICATE SPECIMENS WHICH FAILED IN AN UNKNOWN NUMBER OF CYCLES.

Figure 9.31. Fatigue life of graphite fiber reinforced cement[73]

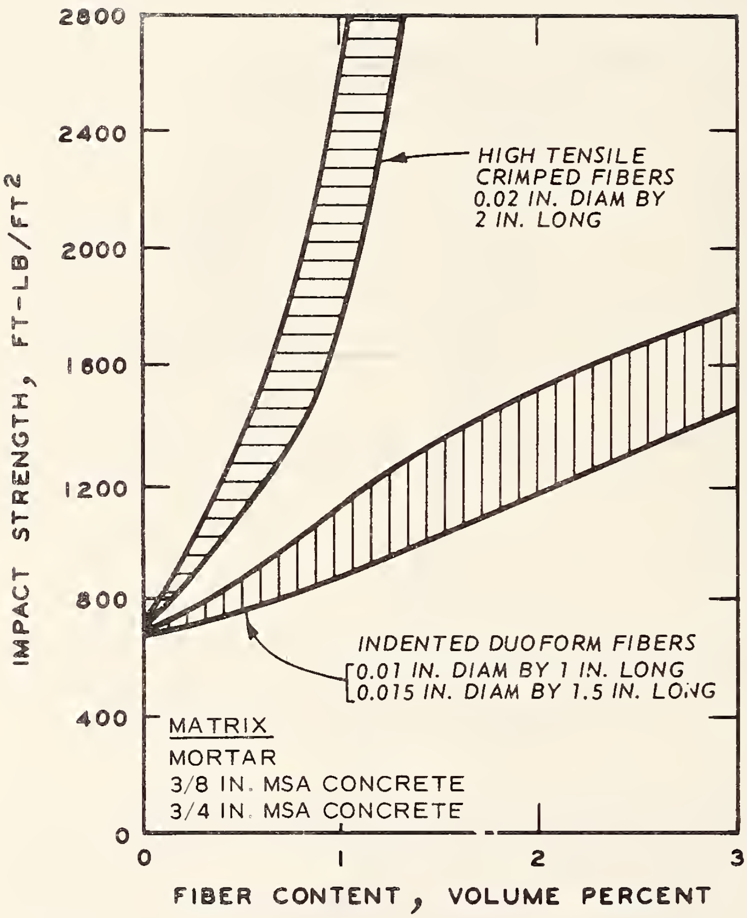


Figure 9.32. Impact strength of steel fiber reinforced mortar and concrete[134]

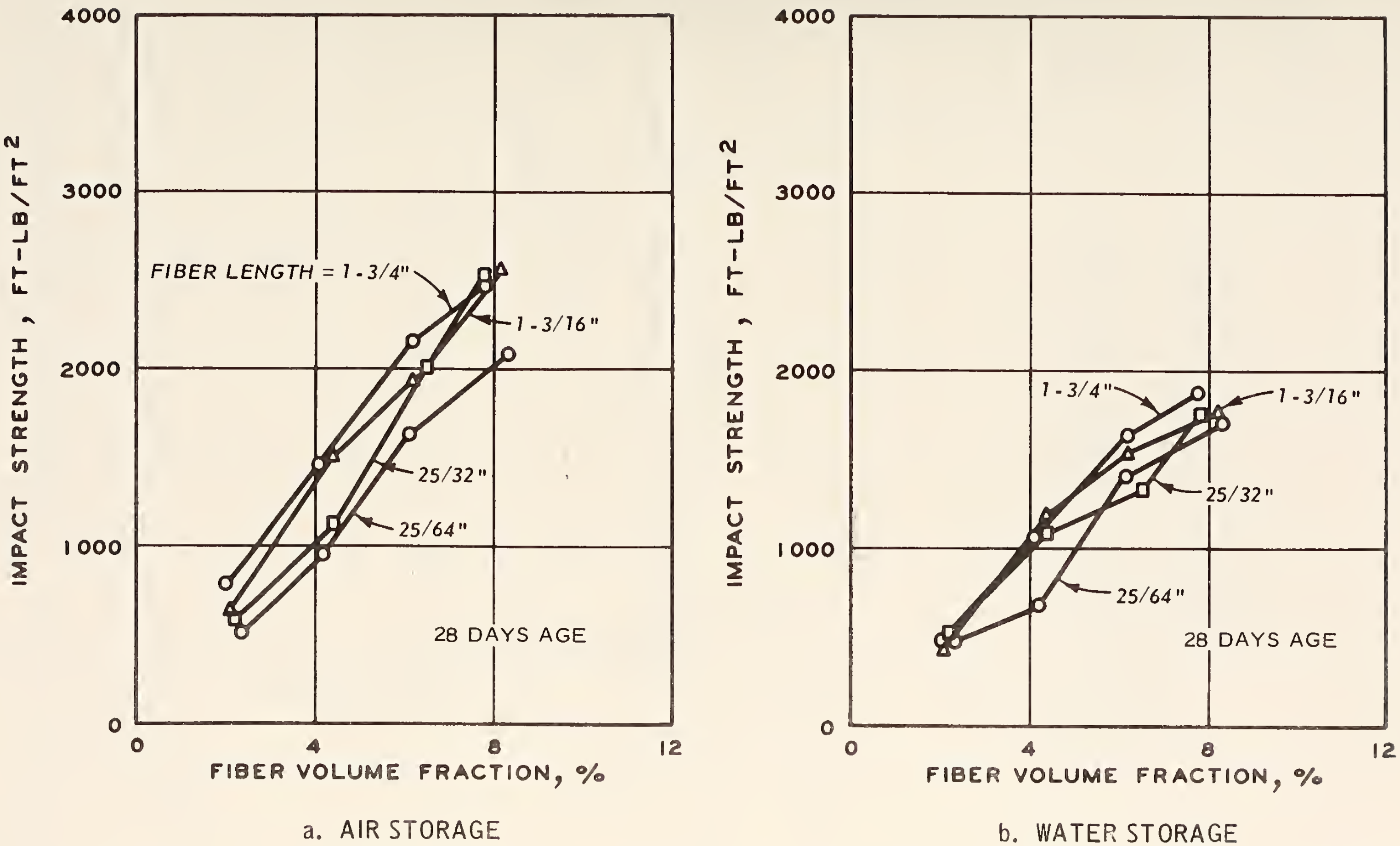


Figure 9.33. Relation between fiber volume fraction and impact strength of glass fiber reinforced cement composites[15]

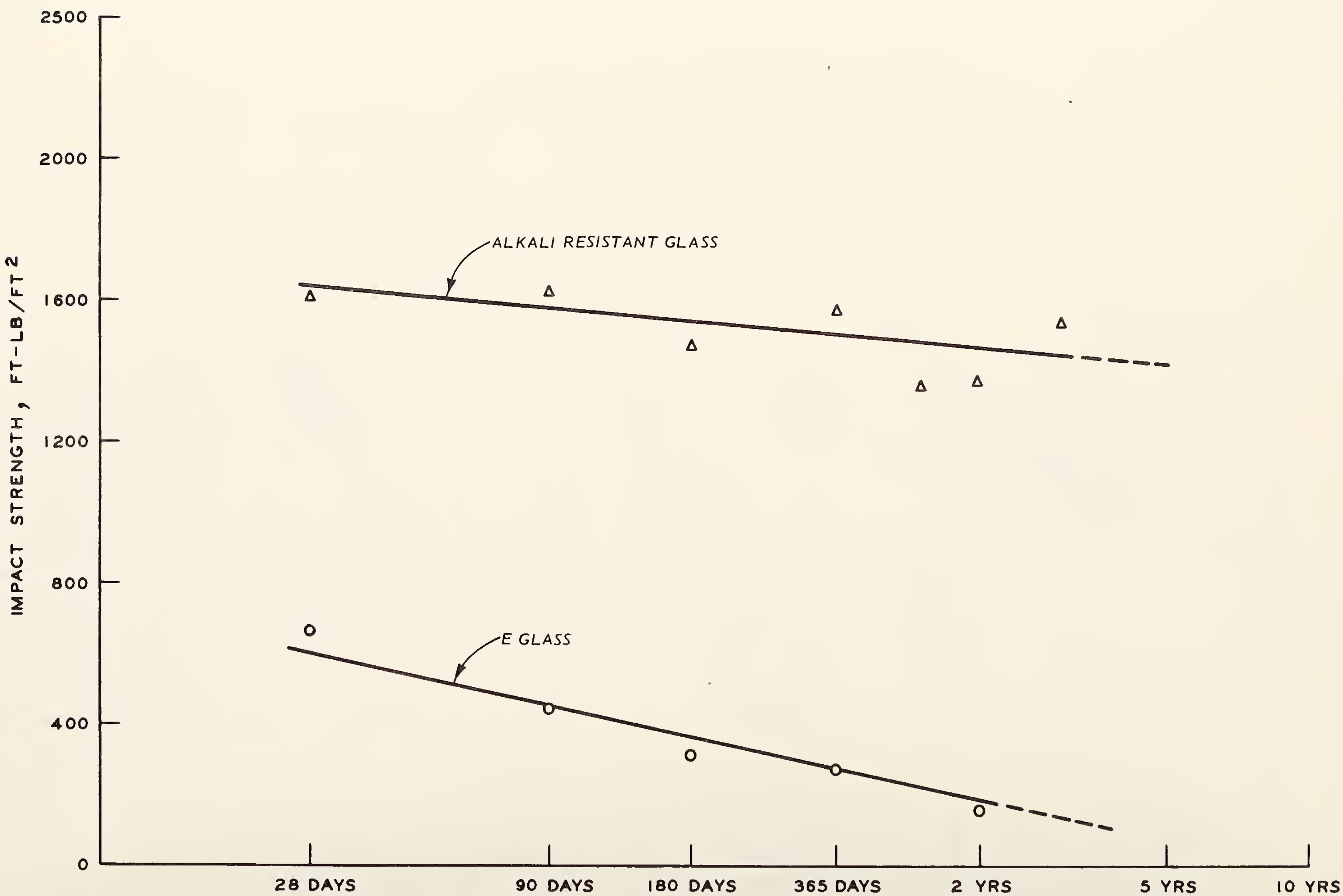


Figure 9.34. Durability of impact resistance of E-glass and AR-glass fibers in an ordinary Portland cement matrix[541]

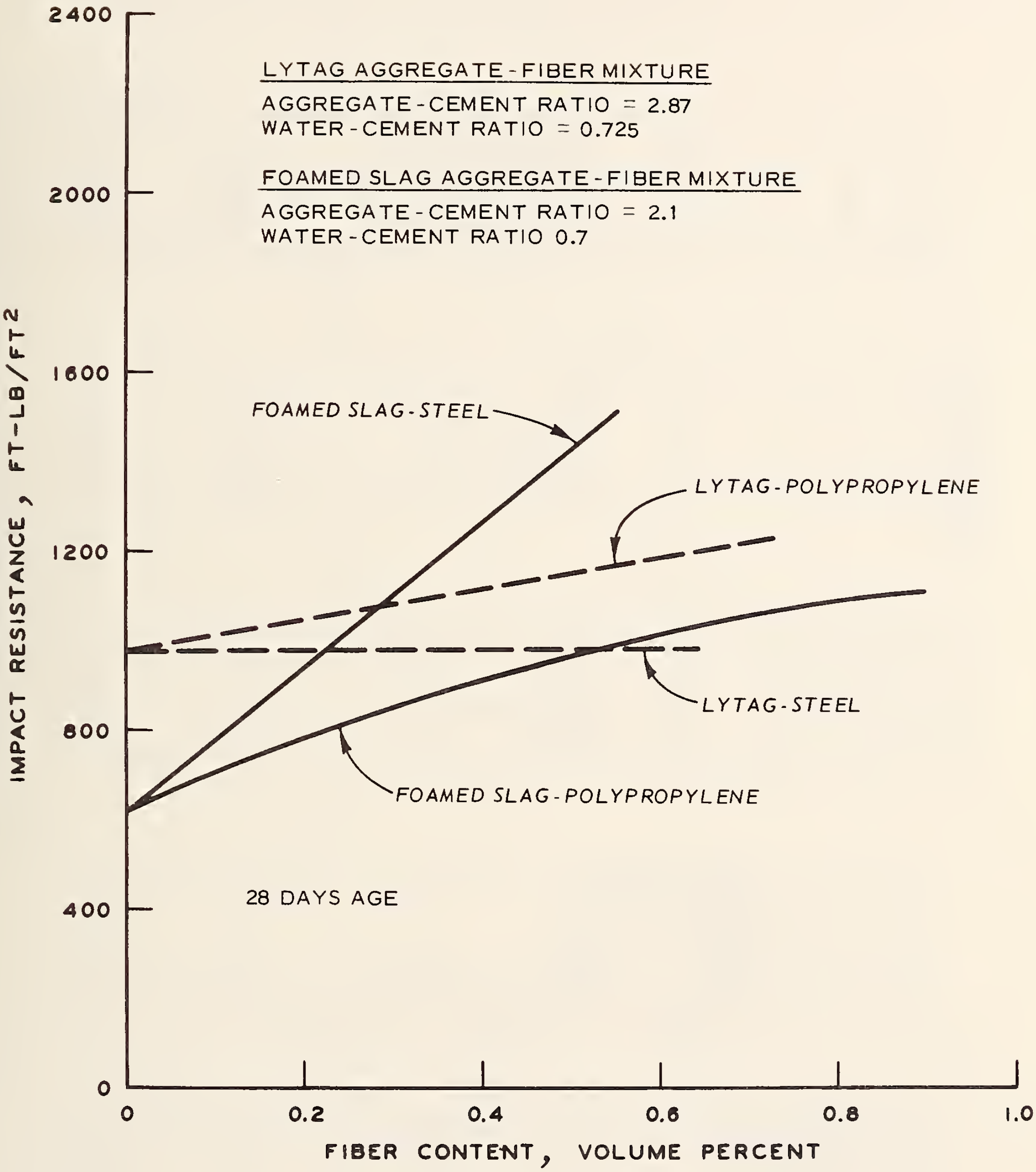


Figure 9.35. Impact resistance versus fiber content relation for lightweight aggregate concrete[475]

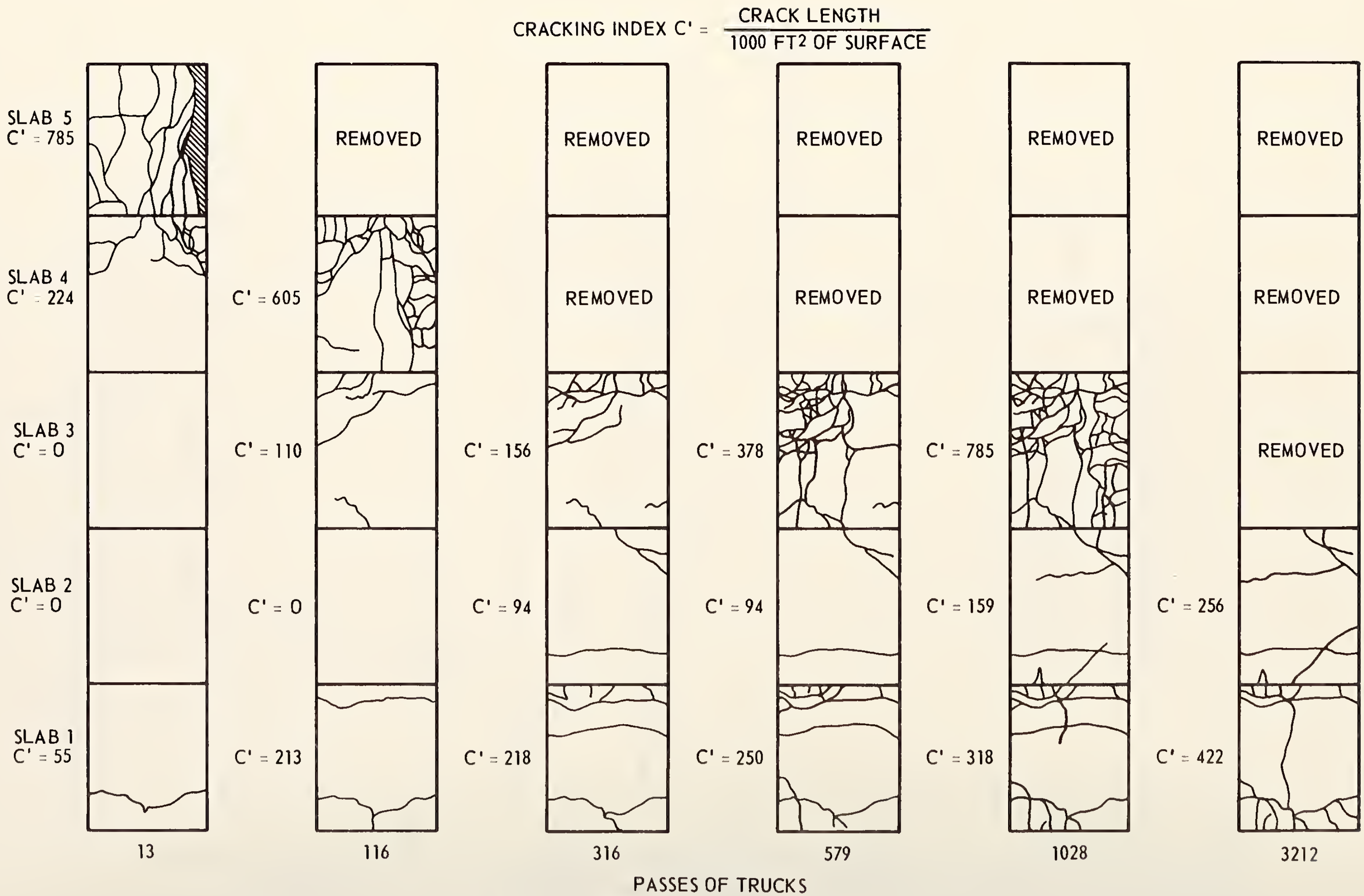


Figure 9.36. Crack development in glass fiber reinforced concrete slabs under 85 kip single axle test loads[79]

CHAPTER 10

POLYMERS IN CONCRETE

INTRODUCTION

10.1 BACKGROUND

Polymer portland cement concrete and polymer concrete have been investigated in various countries since the early 1950's. Large-scale research on polymer-impregnated concrete was undertaken in the United States in 1966. Study and/or use of concrete composites which contain polymers now involves a large number of Government agencies, industrial institutions, universities, and private commercial firms around the world. The rapid advancement of technology in the field of concrete composites which contain polymers is continually producing refinements in components and processes used and in the understanding of properties and performance of the composites. Applications for the composites are expanding at a particularly rapid rate. The information contained in this chapter is an abbreviated view of the total state-of-knowledge on this subject. Notation is contained in Appendix A. ACI Committee 548* is preparing a detailed review of these subjects which should be available in late 1976 or early 1977. Much of this chapter has been extracted from that review.

10.2 DESCRIPTION OF MATERIALS

The utilization of monomers and polymers in concrete is done for the purpose of substantially altering some property of the final hardened concrete material. Small quantities of polymer are sometimes added to modify the properties of fresh concrete. These materials are more commonly referred to as chemical admixtures and are not considered in this chapter.

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*American Concrete Institute, Detroit, MI.

The concretes which utilize polymers to form composite materials have been generally categorized as polymer-impregnated concretes (PIC), polymer-portland cement concrete (PPCC), and polymer concrete (PC). The following definitions describe these materials and other tems commonly used throughout this chapter.

10.2.1 Polymer-Impregnated Concrete. Polymer-impregnated concrete (PIC) is a precast portland cement concrete impregnated with a monomer which is subsequently polymerized *in situ*.

10.2.2 Polymer-Portland Cement Concrete. Polymer-portland cement concrete (PPCC) is a premixed material in which either a monomer or polymer is added to a fresh concrete mixture in a liquid, powdery, or dispersed phase, and subsequently cured and if needed, polymerized in place. The term polymer cement concrete (PCC) has also been used in the literature to refer to this same material definition, but the PCC term is also commonly used in highway work to describe a portland cement concrete pavement.

10.2.3 Polymer Concrete. Polymer concrete (PC) is a composite material formed by polymerizing a monomer and aggregate mixture. The polymerized monomer acts as the binder for the aggregate.

10.2.4 Monomer. A monomer is a molecular species (usually organic) which is capable of combining chemically with molecules of like kind, or with other monomers, to form a very high molecular weight material known as a polymer.

10.2.5 Polymer. A polymer consists of repeating units derived from monomers which are linked together in a chain-like structure. The chemical processes through which these linkages occur is known as polymerization. If only one type of monomer molecule is used to form a polymer, the process is called homopolymerization and a homopolymer results. If more than one chemical species is used as a monomer, the process is called copolymerization. The properties of the copolymer may be varied by controlling the type and degree of copolymerization.

10.2.6 Emulsion. An emulsion is a substantially permanent two-phase mixture made up of very fine particles of a solid or a liquid (the dispersed phase) suspended in a nonsolvent liquid (the continuous phase). The suspension is usually stabilized by small amounts of additional substances known as emulsifiers which modify the surface tension of the particles to keep them from coalescing.

10.2.7 Latex. A latex is an emulsion in which a polymer is the dispersed phase and water is the continuous phase. A latex usually has the appearance of a milk-like fluid.

POLYMER-IMPREGNATED CONCRETE

The basic method of producing PIC consists of the selection of the monomer, fabrication of precast concrete specimens, oven-drying, saturation with monomer, and *in situ* polymerization. The requirements for these steps, the mechanicaly properties of PIC, and some pavement related applications are reviewed in the following sections.

10.3 MONOMERS

The selection of suitable monomers for PIC is based upon the impregnation and polymerization characteristics, availability and cost, and the resultant properties of the polymer and the PIC. Most monomer investigations have been with vinyl monomer systems. Properties of some monomer and polymer systems are listed in Tables 10.1 and 10.2. These monomers were investigated for normal temperature applications of PIC.[1-5] Under normal conditions of temperature and pressure, monomers can be either gases (e.g., vinyl chloride), liquids (e.g., methyl methacrylate, or solids (e.g., acrylamide). Liquid type monomers are most adaptable to impregnation of precast concrete, although gaseous monomers have been used.[3,4]

10.3.1 Monomer Viscosity. The rate and degree of monomer penetration through concrete depends on the density and pore structure of the concrete, and on the viscosity of the monomer.[1] Table 10.1 lists some common liquid monomers of low viscosity at ambient temperature, which are suitable for complete impregnation. Precast concrete can be successfully impregnated with higher viscosity monomers (greater than

20 centipoises or 20 millipascal-seconds),[6,7] although it is usually advantageous to reduce viscosity by suitable blends with low viscosity comonomers, e.g., methacrylate polymers.[8] The viscosity may be increased by dissolving a polymer in the monomer.[4] The viscosity of methacrylate (MMA) solutions can be varied greatly in this manner.

10.3.2 Monomer Volatility. In the selection of a monomer for impregnating precast concrete, considerations must be given to its volatility (Table 10.1) for safety and processability. The high vapor pressure of vinyl chloride, for example, requires special precautions in handling. Considerations must also be given to the effect of curing temperature on vapor pressures, since monomer depletion on the surface of the specimen may occur[1,3,9] due to evaporation. Low viscosity monomers tend to have low boiling points while high boiling monomers are more viscous.

10.3.3 Chemical Stability of Monomers. Monomers are generally supplied containing an inhibitor. Inhibitors are chemical compounds which react with free radicals and oxidation products to prevent premature polymerization and to maintain the purity of the monomer. Over a period of time, the inhibitor is consumed and its concentration gradually decreases. The rate of inhibitor consumption also depends on the storage temperature and cleanliness of the storage facility. Stored monomer should be tested regularly for inhibitor content, and inhibitors should be added if the level drops appreciably below the original level. Concentrations of inhibitor can be determined by various methods.[3,10] During polymerization of the monomer, an induction period is observed, which represents the period during which the inhibitor is consumed.

10.4 POLYMERS

10.4.1 Thermal Properties. One of the most important performance characteristics of polymers is the marked dependence of their properties on temperature within their useful temperature range. Thermoplastics retain their useful properties at temperatures below their glass transition temperature (Tg) but lose them at higher temperatures. At generally somewhat higher temperatures, the polymers begin to thermally decompose. Thermoset polymers do not exhibit a Tg and retain their structural properties up to the thermal decomposition temperature. The thermal decomposition temperature of some typical vinyl polymers ranges from about 80C for polyvinyl chloride (PVC) to 260C for poly(methyl methacrylate) (PMMA), but is generally about 200C (Table 10.2).

The useful temperature range of a thermoplastic may be raised with the addition of a suitable crosslinking monomer or comonomer having a higher Tg. Tg for several MMA and styrene comonomer systems are given in Table 10.3. It should be noted, however, that these systems may not be stable under all exposure conditions; for example, it has been observed that the 90-10 MMA-diallyphthalate (DAP) and 70-30 MMA-trimethylopropane trimethacrylate (TMPTMA) systems fail under prolonged exposure to moisture at elevated temperatures, whereas under the same conditions the 60-40 styrene-TMPTMA system is stable.

10.4.2 Mechanical Properties. Tensile strengths of polymers are relatively high at temperatures well below Tg. Most glassy, vinyl polymers have a tensile strength in the range of 5,000 to 10,000 psi; compressive strengths are not much greater, ranging from about 11,000 to 19,000 psi. Below Tg, where the polymers are hard, brittle materials, the modulus of elasticity is relatively constant ($\sim$ 100 ksi). Above Tg, the modulus drops to relatively low value ($>$ 100 psi) and the materials exhibit plastic behavior. The same is true for the shear modulus. Other deformation characteristics (e.g., coefficient of thermal expansion and creep strains) are significantly increased about Tg.

Polymers are denser than their monomer. The shrinkage from polymerization may produce detrimental effects on the mechanical properties of the polymer. In the use of polymers for PIC, shrinkage may result in poor bonding between the polymer and the substrate. In addition, the volume change due to shrinkage during the *in situ* polymerization can still leave some voids in PIC so that durability properties may be less than the maximum possible. Built-in stresses and strains can be relieved in thermoplastic polymers by annealing at a temperature close to Tg, with subsequent slow cooling.

10.4.3 Chemical Properties. Generally, polymers are inert materials, unaffected by most reagents. They may, however, be attacked by oxidizing agents or aromatic or chlorinated organic solvents. Absorption of ultraviolet (UV) radiation may lead to eventual degradation of some polymers. Stress cracking is often a problem with many polymers.

10.5 ADDITIVES AND MODIFIERS

The various physical and chemical properties of polymers have been discussed in section 10.4. Various comonomers and other additives to the monomer system are frequently used to modify or produce desired changes in the properties of the polymer.

10.5.1 Plasticizers. Plasticizers are commonly added to monomers to improve the flexibility of inherently brittle polymers such as poly(methylmethacrylate) and polystyrene (PS). Specific examples are the addition of "internal plasticizer" like vinylstearate, butyl acrylate (BA), or dibutylphthalate which co-polymerize with the monomer.[5,10]

10.5.2 Cross-Linking Agents. Cross-linking by means of the addition of an appropriate bi-functional or poly-functional monomer increases the rigidity of the polymer, its resistance to the action of solvents, and its softening point. The amount of change depends on the cross-linking density in the polymer. The cross-linking agent most commonly used in PIC is TMPTMA[1] which is a tri-functional acrylic monomer which can be homo-polymerized or co-polymerized with other vinyl monomers such as MMA or styrene.

10.5.3 Flame Retardants. All polymers will burn, pyrolize, or char in contact with a flame source. It is known, however, that compounds containing certain elements such as chlorine, bromine, phosphorous, antimony, boron, and nitrogen will retard or inhibit combustion. The degree of flame retardancy depends mainly on the amount of the flame-retarding elements in the composition. Flame retardants are divided into two general classes: reactive additives which become part of the polymer structure, and inert additives. There is no significant difference in efficiency between the two types, the dominant factor being the content of flame retarding element. Convenience, availabiltiy, and cost and degree of flame retardancy are factors that determine the most advantageous type. The physical properties of polymers may change significantly as a result of dilution and plasticization of the polymer. Flame retardants are blended with the monomer prior to polymerization in proportions depending upon the desired degree of flame retardancy. The flame retardants react to provide either inert, noncombustible gases (antimony, bromine, nitrogen) or to promote the formation of protective chars (boron, chlorine, phosphorous). Toxic gases may often be generated from flame retardants under these conditions.

10.5.4 Silane Coupling Agents. Silane coupling agents are monomeric, silicon chemicals used to chemically bond organic polymers to inorganic materials such as sand, rock, glass, and metals.[11,12,13] They have the general formula $(HO)_3SiR$ where R is an organic group compatible with thermoplastic or thermosetting resins. Coupling agents have been used occasionally in PIC and more commonly in PC for improvements in strength[5] and for retention of aggregate bond in long-term exposure to moisture.

10.6 PIC FABRICATION

No special procedures are necessary for the preparation of concrete for impregnation. All types of aggregates, cements, and admixtures that can be used in modern concrete technology can be used for PIC. Similarly, curing procedures for strength development are not critical. Table 10.4 shows the results of some experiments on concrete composition and curing.[6] Of course, the final properties of the PIC may vary somewhat, depending on the nature of the materials or curing conditions used. The highest strengths have been obtained with high-pressure steam-cured concrete.[5] It appears preferable to start with a good quality dense concrete, as the polymer loading for full impregnation is much lower than for the more porous poorer quality concrete.

10.7 CONCRETE DRYING REQUIREMENTS

The strength and durability properties of PIC are strongly affected by the fraction of the porosity of the cement phase which is filled with polymer. If maximum improvements are desired, it is necessary to remove as much water from the precast specimens as possible prior to impregnation in order to maximize the available porosity.

Drying temperatures up to 750F have been evaluated.[14,15,16] The amount of water removed in a given concrete specimen is a function of both drying time and temperature. Recent work[6] indicates that a drying temperature of 230F may not be high enough to remove all the water. It has been found[4] that specimens dried at temperatures from 302F to 482F and subsequently impregnated with MMA, exhibited a slight decrease in compressive strength with increased drying temperature. Surface temperatures up to 750F have been used successfully with slabs and on small bridge deck sections in order to get deep penetrations of monomer.[15,16] A drying temperature of 302F seems to be optimum. At this temperature the drying rates are reasonably fast ($\sim$ 24 hr for a 3-ft inside diameter x 6-ft-long x 4-ft-thick pipe), and a high quality product is produced.

These studies have not specifically evaluated the effects of heating and cooling of large specimens which may be susceptible to cracking. For large specimens, special care and control of heating and cooling rates, and the selection of heating temperature, may be required to prevent or minimize cracking the concrete.

10.8 MONOMER SATURATION TECHNIQUES

10.8.1 Full Impregnation. Experiments to determine the conditions required to fully impregnate concrete have shown that process parameters such as degree of dryness and vacuum, soak pressure, and soak time, all have an effect on the strength of PIC.[6,7] Complete saturation of concrete using modest pressures (10 psig) is accomplished only after prior evacuation. Without evacuation, soak pressures of 100 psig are required in order to approach saturation in a short period of time ($\sim$ 60 min.).

Studies[6] have shown that good results can be obtained on good quality dense concrete specimens having a cross-section of up to 12-in. Techniques for accomplishing this are detailed in Reference 6. Full impregnation of pavement slabs on grade is not very feasible, however.

10.8.2 Partial Impregnation. Soaking unevacuated samples at atmospheric pressure results in partially saturated specimens and, therefore, somewhat lower strengths. In general, overnight soaking of a dried, 5000 psi, small-sized concrete specimens in MMA will result in filling only 70 to 80 percent of the voids that can be filled when the concrete is first evacuated. Thus, compressive strengths are not likely to exceed 15,000 psi to 17,000 psi.

Techniques for the partial impregnation of concrete include complete immersion in monomer and application of monomer to just one surface.[4,5,7,14] These impregnations have been with soaking under pressure and soaking at atmospheric pressures. Atmospheric soaking with low viscosity monomers has produced fairly good results with impregnation depths of up to several inches. Pressure soaking produces better results with deeper impregnation and higher polymer loadings. A study on soaking dried concrete specimens in monomer at atmospheric pressure showed polymer loading and depth of penetration was proportional to the log of soak time.[1] Other studies have shown a linear dependence of penetration rate on the square root of time[15,17,18] and on the applied pressure.[15] For deep penetrations from one side, where the use of applied pressure may be advantageous, the use of a pressure mat appears to have promise.[19]

The use of higher viscosity monomers results in a more uniform depth of penetration and greater control,[5,16] but with these materials, positive pressures are required. A soak time of ~ 5 hr at 100 psig pressure was required to obtain a 3/4-in. penetration with a 67.5-wt. percent styrene-32.5 wt. percent polyester mixture (viscosity 10 cP). Surface impregnation of dried concrete bridge decks have been made by soaking with MMA monomer at atmospheric pressure.[20] The studies have shown that it is necessary for the bridge deck to be dry in order to achieve a good impregnation. Care is required in drying the bridge decks, as excessive drying temperatures and drying times may result in some cracking of large slabs. Using this method, concrete has been impregnated to depths of 1-1/2 to 2 in. with soaking periods of from 8 to 12 hr.

10.8.3 Encapsulation Techniques to Reduce Monomer Losses. Care must be taken to minimize monomer evaporation and drainage losses from the concrete during the polymerization reaction. Evaporation is a problem when high vapor pressure monomers such as MMA are used. Monomer drainage losses become appreciable when low density concretes are impregnated. The following methods for reducing these losses have been investigated:[5,7,14,20,21,22,23]

a. Wrapping monomer-saturated specimens in polyethylene sheet or aluminum foil.

b. Encapsulation of the specimen in a form during impregnation and polymerization.

c. Polymerization with the monomer-saturated specimens immersed in water.

d. Impregnation with monomer followed by a pre-polymer dip prior to wrapping the specimens.

Only item b is directly applicable to the *in situ* polymerization of cast-in-place pavements although underwater polymerization may have some promise as it appears to be the most practical for large-scale application. The method has been used successfully in conjunction with radiation and thermal-catalytic polymerization.[6] If the water is saturated with monomer prior to use, very little surface depletion is observed. The results[14,21,22] also indicate that underwater polymerization does not have any detrimental effects on the properties and can produce specimens with highly reproducible polymer loadings.

It should be noted that the underwater polymerization process also produces some polymer in the water which may adhere to the walls of the impregnator and collect in the valves and piping. Less polymer is formed in the thermal-catalytic process probably due to the decreased solubility of MMA in water at the elevated temperature. The problem can be minimized by designing the vessel to drain all the excess monomer and installing filters in the water system.

10.9 POLYMERIZATION

There are three general methods for the bulk polymerization of monomers currently being used in PIC. These are the radiation, thermal-catalytic, and promoter-catalyst techniques. The selection of a particular process will depend on its particular advantages for a specific application and evaluation of the effects of (1) drainage and evaporation losses from the concrete during the polymerization; (2) safety problems associated with the storage and reuse of large quantities of monomer and catalyst; and (3) the economics of the entire process.

10.9.1 Thermal-Catalytic. The simplest method is thermal-catalytic polymerization through the addition of small amounts of a compound which will generate free radicals on heating. The following commercially available compounds have been used in forming PIC:[1-5] benzoyl peroxide (BzP), azobis (isobutyrolnitrile) a-tert butylazo isobutrolnitrile, tert-butyl per-benzoate and methylethylketone peroxide. These compounds decompose at different rates or over a range of temperatures to generate free radicals. The section of type and concentration of initiator and the optimum polymerization temperature are important in the production of a uniformly good quality PIC. BzP catalyst is well suited for most vinyl monomers, such as methyl methacrylate and styrene, since it decomposes well below the boiling point. However, there is some controversy over the use of BzP in PIC. Some investigators have reported difficulties in achieving a uniform polymerization.[5] BzP is also subject to induced chemical decomposition, which increases the risk of an accidental bulk polymerization of the catalyzed monomer when it is stored. These problems have not been encountered with azonitrile compounds, and some prefer these compounds for PIC. A higher temperature catalyst such as tert-butyl perbenzoate is more effective with higher boiling point monomers like diallyl phthalate.

It should be noted that organic peroxides are shock sensitive, highly reactive, and may decompose explosively. Special precautions must therefore be taken when dealing with large amounts of monomer-peroxide mixtures. The use and handling of chemical catalysts should be in accordance with procedures recommended by the manufacturer.

The thermal-catalytic process has been used extensively for preparing PIC and appears the most practical for present-day use. The process can be performed in air or under water. Several catalysts mentioned above have been used in this method. The primary advantage of the thermal-catalytic polymerization method is that the polymerization rates are very rapid and, therefore, processing times are short. Relatively simple elctric ovens, water, or raw steam can be used as a heat source.[6,14,20,21,23] A disadvantage is that the chemical initiator must be dissolved in the monomer prior to introducing the mixture into the concrete. In a commercial operation of almost any size, this will involve storing and handling of large batches of monomer containing chemical initiator. Although potentially dangerous, the use of relatively stable azotype initiators in conjunction with established safety practices can reduce the hazards to manageable levels.[5,6] Catalyzed MMA monomer has been safely stored for periods greater than one year.

10.9.2 Promoted-Catalytic. Decomposition of organic peroxide catalysts can be initiated by promoters or accelerators instead of temperature. These compounds are reducing agents which induce the decomposition of the peroxides. Thus, polymerization reactions can take place at ambient temperature. Several promoters which are commonly used are methyl anilines, dimethyl-p-toluidine, cobalt napthenate, and meraptane. Promoter-catalyst systems can induce polymerization at a temperature of 5C or lower[6] and are well suited for PC. However, because polymerization begins immediately on adding a promoter to the monomer-catalyst system, its use in PIC would be restricted to shallow impregnations.

The primary advantage of a promoter-catalyst system is that the polymerization can be initiated at ambient temperature without the need for an external source of energy. Disadvantages are the difficulties in obtaining predictable polymerization times and in being able to match the monomer saturation time with that of the onset of polymerization. The process, therefore, does not appear suitable for producing fully impregnated concrete but may have application for some partially-impregnated concrete.

The techniques for using promoter-catalyst systems as a means of producing PIC have not as yet been demonstrated on a large scale. However, the system is being used in field applications.[6,20,21,24,25] In all of the work, the materials were mixed immediately before application. Methods of application include injection, spraying, and use of paint brushes and rollers.

10.9.3 Radiation. The production of free radicals during initiated polymerization can also be achieved by the use of ionizing radiation such as gamma rays emitted by cobalt-60. Absorption of the radiation energy by the monomer results in secondary processes including the production of free radicals.[13] The rate of polymerization varies with the different monomers under constant radiation and temperature conditions. The polymerization rate is dependent upon the square root of the intensity,[26,27] but at very high radiation intensities it reaches a limiting value. An important advantage of radiation curing is that chain reactions can be initiated at room temperatures or lower. Lower temperature polymerization increases the chain length of the polymer and tends to reduce the amount of monomer lost by evaporation before complete polymerization takes place, particularly when monomers of high vapor pressures are used. Since catalysts and promoters are not required for this process, the inhibited monomer can be used directly as it comes from the manufacturer. However, some monomers require high radiation doses and polymerize slowly.

The radiation-induced polymerization of monomers in concrete has been performed in air and under water.[1,3,4,6,27] The principal advantage of the process is the elimination of a catalyst which therefore allows essentially unlimited storage and reuse of monomer. Also, the polymerization can be initiated at room temperature and at a uniform rate within relatively thick concrete sections. Detrimental features include the high cost of radiation sources, the necessity of massive biological shielding, and the low polymerization rates. The latter, when combined with the radiation attenuation due to the thick sections and high density, results in large radiation requirements and long processing times.

10.10 SAFETY CONSIDERATIONS

The handling of monomers, particularly in large volumes, requires rigid safety controls. Monomers are shipped from the manufacturers containing sufficient inhibitor to maintain safety during shipping and storage. Manufacturers recommend proper storage temperatures, toxicity limits, and other precautions pertinent to safe handling of the monomers. A decrease in inhibitor concentration with time or from contact with concrete indicates that the monomer is becoming unstable and may ultimately polymerize. Therefore, the monomer must be tested periodically to determine the inhibitor concentration. Inhibitor concentrations can be measured by various chemical and colorimetric methods, but these methods may not give satisfactory results after the monomers have come in contact with concrete, due to impurities and discoloration. A peak exotherm method has been used successfully for determining the inhibitor concentration of monomers after contact with concrete.[3]

The greatest potential hazards exist when a catalyst has been added to the monomer as in the thermal-catalytic processing method. In this case, the catalyst concentration is well in excess of the inhibitor concentration and rapid premature polymerization could occur under unfavorable conditions. Catalyst concentration can also be measured by the peak exotherm method.[3]

Many monomers have low flash points and are, therefore, Class I flammable liquids as defined by the National Fire Protection Association. Precaution should, therefore, be taken to prevent exposure to flames, sparks, or other ignition sources. All containers should be electrically grounded. Electrical devices in areas where monomers are handled should be explosion-proof, and spark-proof tools should be used.

10.11 MECHANICAL PROPERTIES OF PIC

10.11.1 Appearance. Polymer-impregnated concrete looks very much like conventional concrete. Apart from some surface coating of polymers which may be present, PIC is indistinguishable from plain concrete on casual observation.

10.11.2 Strength.

10.11.2.1 Mechanism of Strengthening. High strengths in compression, tension, and flexure can be achieved in PIC with increases being as much as four times that of control specimens. These improvements stem largely from the fact that water or air-filled pores are replaced by a load-bearing polymer phase. The addition of polymer to the hardened concrete causes healing of micro-fractures and produces improved bond between cement paste and aggregate. Therefore, the final strength of PIC depends on (1) the extent of the impregnation and filling of pores; (2) the type of polymer and its ability to carry stress; and (3) the degree of conversion of monomer to polymer during polymerization.

An empirical relationship (Eq. 10.1) has been developed[28] to predict the compressive strength of PIC

$$S = S_m V_m + A\ B\ S_d V_d \tag{10.1}$$

where S, S_m, S_d are the strengths of PIC, cement paste, and aggregate, respectively; V_m and V_d are the volumes of cement paste and aggregates; A is a constant derived theoretically and experimentally, and equals 0.35, and B is a factor relating to hardening between cement and aggregate. B = 1 for perfect bonding and is $\sim$ 0.5 for normal concrete. Equation 10.1 is a first attempt to quantitatively predict the strength of PIC. Its main limitation is that it relies exclusively on increases in strength of the cement-aggregate bond to account for increases in the strength of PIC and takes no accounts of other mechanisms. Such an approach can lead to erroneous conclusions; for example, a PIC made from a more porous concrete (before impregnation) would have a lower compressive strength due to lower values of S_m and S_d. However, studies[4,14,29] have shown this not to be the case for MMA-impregnated concrete. Flajsman, et al.,[30] used this property to advantage by using a highly porous mortar to promote impregnation without the application of vacuum or pressure. In contrast, Whiting, et al.,[31] found the compressive strength of a diluted epoxy-resin PIC to depend on the w/c ratio of the original concrete.

10.11.2.2 Effect of Proportioning. Minor changes in aggregate composition or gradation do not affect the properties of PIC.[5] The most noticeable effect of concrete properties is that the more porous concretes require more monomer for complete impregnation, which will tend to affect the economics in producing PIC more so than the final properties of the PIC. When a nonporous aggregate is used, the improvement in strength depends mainly the degree to which the cement paste is impregnated.[5,22,32,33] Thus, it follows that factors such as the cement content and w/c ratio control the amount of monomer required for complete impregnation. However, the strength of the PIC will be largely independent of the quality of the initial concrete provided full impregnation is achieved. Thus, PIC produced from a low-quality concrete with a high w/c ratio may approach or be equal in quality to PIC produced from a good quality, low w/c concrete but at the expense of a higher polymer loading for the more porous concrete.

Impregnation of lightweight concretes can lead to strength increases over controls similar to normal weight impregnated concretes and yet still retain weight advantages over regular concretes.[22,34] Very light concretes will have high polymer loadings due to partial impregnation of the aggregate porosity.[21,35] Loadings for MMA have ranged from 31 volume percent for a foamed glass concrete to as much as 55 volume percent for a vermiculite aggregate concrete; normal weight concrete typically will accept 12 to 15 volume percent of polymer. Nonimpregnated concrete having strengths of less than 1000 psi have generally been improved to between 3000 and 5000 psi, when impregnated, and in some cases, the strength loss exceeded 6000 psi.[34] It has been suggested that the improvement in strength of lightweight concretes depends to a large extent on the polymer-aggregate interaction.[35] Thus, PIC's from lean lightweight concretes may show a larger improvement factor than do PIC's from lean normal weight concretes.

10.11.2.3 Effect of Curing. It further follows that the extent of moist curing prior to impregnation will not affect the properties of PIC, although it will again change the amount of polymer required[4] for full impregnation. However, concretes subjected to both high and low pressure steam curing prior to impregnation generally result in higher strengths for PIC at lower polymer loadings[1,5,36] than comparable concretes moist cured at room temperature. Strengths in excess of 30,000 psi have been achieved. These high strenghts were attributed to the fact that steam-cured concretes have larger pore sizes (although similar total porosity) leading to a more efficient polymer loading of the concrete.

10.11.2.4 Effect of Monomer and Polymers. The ability of different monomers to provide high strength PIC depends on a number of factors:

a. The efficiency of impregnation.

b. The formation of a continuous polymer phase.

c. The mechanical properties of the polymer.

Monomer requirements have been discussed in detail.[1,3] Properties of PIC fabricated from common monomers are given in Table 10.5. Several monomer systems, capable of cross-linking and providing good properties at high temperature have been studied.[3,4] In addition, Solomatov[37] has impregnated concrete using urea-formaldehyde resin and chlorovinyl resin in dichloroethane. Epoxy-styrene and polyester-styrene resins have been used for partial impregnation.[22]

The extent to which a monomer can fully impregnate the void space of concrete, under ideal processing conditions, depends primarily on its viscosity. Thus, other methacrylate esters such as stearyl, isobornyl, and isodecyl, did not produce as good a PIC as did MMA[38] since polymer loadings were limited by the higher viscosities on the monomers (Table 10.6). Søpler, et al.,[14] have shown the strength of concrete partially impregnated with MMA is a function of the polymer loading. Excessive evaporation of monomers will also have an important influence on monomer loadings and precautions must be taken during processing to prevent this.

The mechanical properties of the polymers may also affect the performance of PIC. Vinyl polymers have generally lower strengths than thermosetting resins, but the high viscosity of the latter precludes their use for complete impregnation of concrete; they can, however, be used for partial impregnation. In situ polymerization to form PVC and polyvinylidene chloride results in a powder rather than a continuous glassy phase.[4] This is probably the reason why PIC containing these polymers show little improvement in properties. Acrylonitrile has a tendency also to form a powder on polymerization since the polymer is insoluble in the monomer.[1,3] Such a phase is not always observed in PIC, but it is possible that a mixture of a powder and glassy phase may form, thereby limiting the the strength of acrylonitrile-PIC. The addition of TMPTMA to MMA decreases the flexural strength of PIC.[4,38] This effect may be caused by increased brittleness of the co-polymer.

10.11.2.5 Effect of Polymerization. Studies have shown that polymerization by gamma irradiation results in a PIC with strengths greater than that produced by thermal-catalytic polymerization.[3,4] The difference between methods depends on polymer type (Table 10.7) and is most pronounced with styrene and acrylonitrile. Radiation has a neglible effect on the strength of dried and undried concrete containing no monomer.[5] This difference between the two methods of polymerization may well be traced back to differences in the completeness of polymerization of the monomer, although no studies have been made to check this point.

10.11.2.6 Effect of Temperature. At temperatures above T_g of the polymer, the PIC would be expected to lose most of its strength attributed to the polymer-impregnation. However, at temperatures below T_g, some temperature dependence of strength has been observed. Table 10.8 shows the effect of temperature on the compressive strength of PIC prepared with a high temperature polymer for temperatures ranging from -10F to 350F. This polymer, (60-40) styrene-TMPTMA has a T_g of 417F.[4,5,6,39] The prolonged exposure at elevated tempeartures also slightly reduces strength. Similar effects have also been observed for tensile and flexural strengths. Other investigators[40,41] report that if PIC specimens are heated to 392F and then cooled, some strength gain may be observed. Heating PIC at temperatures above 392F results in progressive evaporation of the polymer with concomitant loss of strength. Drying concrete prior to impregnation at temperatures higher than 392F was first reported[3] to have a detrimental effect on the strength of PIC containing 5.5 volume percent MMA. Subsequent investigations[4] have not confirmed the earlier work and effects of drying temperatures on strength may not be significant.

10.11.3 Modulus of Elasticity. Increases in both elastic and flexural moduli accompany the strength increases (Table 10.5) of PIC over unimpregnated concrete; when high-strength concrete with a modulus of 5000 ksi was impregnated, elastic moduli greater than 7000 ksi could be obtained.[4] The effect of polymerization method on moduli of PIC seems to be variable.[3,4]

With partially impregnated concretes the moduli of elasticity are not significantly different from the control; they may be slightly higher or lower. However, the stress-strain curve was found[14,41,42] to be linear up to 70 to 75 percent of ultimate strength to deviate from linearity at failure by only 10 to 15 percent.

10.11.4 Dimensional Changes.

10.11.4.1 Thermal Expansion. Since the percentage of polymer in PIC is small, differences in thermal expansion between different PIC's and between untreated concretes should not be large. Generally, PIC shows a slight increase in thermal coefficient of expansion (up to about 25 percent) over untreated concrete.[3,4] Tazawa and Kobayashi[43] have shown that mechanical properties of PIC are influenced by self-stress generated in the material, possibly due to shrinkage on polymerization or differential thermal expansion between polymer and concrete. Annealing or addition of plasticizer are means of relieving this stress.

10.11.4.2 Polymerization Shrinkage. Relatively large shrinkage strains (up to 300 millionths) are observed in concrete during polymerization.[29] This is due to a volume contraction from polymerization[31] which may be up to 20 percent. Volume changes on polymerization may result in unfilled void spaces within the PIC and possibly residual stresses in the polymer phase. The effects of shrinkage have not been studied.

10.11.4.3 Creep. Compressive and tensile creep data for MMA-PIC and conventional unimpregnated concrete under several loading conditions are given in Table 10.9. The low creep deformation exhibited by PIC appears for the most part to be due to the polymer impregnation, and to a lesser extent a result of drying the concrete prior to impregnation.

A comparison of creep for MMA-PIC, conventional (undried concrete), and oven-dried concrete under 800 psi compressive load is given in Table 10.10.

After 1500 days sustained loading, the MMA-PIC shows a slight negative creep, the oven-dried concrete shows creep of 0.096 millionths/psi, and the undried conventional concrete shows a creep of 0.291 millionths/psi. It should be noted here that the loading stress/ultimate strength ratio for the PIC specimens (about 0.04 for PIC) is much lower than that for the unimpregnated concrete (about 0.16).

The effects of temperature on the creep of PIC designed for high temperature service are shown in Table 10.11. These specimens were made with a special high strength concrete and a concrete of normal strength. In the case of the specimens made with the special high strength concrete, a comparison of creep may be made on the basis of approximately equivalent loading stress/ultimate strength ratios, with unimpregnated concrete loaded at 2313 psi and PIC loaded at 7000 psi. These loadings are at a loading stress/ultimate strength ratio of about 0.37.

10.11.5 Durability. Durability of PIC to most forms of environmental attack is significantly improved over that of untreated concrete. This can be attributed in part to increased strength, but primarily to filling of the pore system in concrete with polymer. This will prevent ready ingress of moisture and deleterious substances and is reflected in the much lower permeabilities and water absorption observed for PIC (Table 10.12). Durability performance of PIC is summarized in Table 10.13. Generally, thermal catalytic polymerization results in slightly more durable PIC.[1]

Partially impregnated concrete can be used to provide improved durability[4] if high strength is not a major consideration. With polymer loadings of 1.5 to 2.5 wt percent, corresponding to about 1/16 to 1/4 in. impregnation, significant improvements in durability can be achieved. However, results are often erratic due, most probably, to failure to achieve a complete shell of impregnated concrete at the surface of specimens. Surface impregnated slabs have shown improved durability.[20]

10.11.5.1 Freeze-Thaw Resistance. Generally, all PIC containing common monomers showed great improvement of freeze-thaw durability[1,44] (Table 10.13). Specimens were exposed to temperature cycles from -12C to 21C and a 25 percent weight loss was used to indicate failure.[3] PIC specimens also had excellent resistance to surface freeze-thaw conditions.[38] Partially impregnated specimens containing 1 to 3 weight percent of polyester-styrene or epoxy-styrene went more than 1500 cycles without failure when impregnated 1/4 in.

10.11.5.2 Chemical Resistance. PIC generally shows excellent resistance to sulfate attack and to 15 percent HCl. Attack by 15 percent H_2SO_4 is a more severe exposure condition and even PIC specimens failed under these exposure conditions (25 percent weight loss) relatively quickly, although their performance is superior to that of untreated concrete. Partially impregnated concrete also shows some improvements in resistance to chemical attack.[4]

10.11.5.3 Temperature and Fire Resistance. PIC generally shows a gradual reduction in strength with increased temperature.[6] The maximum temperature limit for retention of useful structural properties is primarily governed by the glass transition temperature of the polymer, and ultimately by thermal degradation of the polymer. Most vinyl polymers have a glass transition temperature of about 200F.[3] Cross-linked and thermosetting polymers may have higher useful temperature limits.[45] A 60 percent styrene-40 percent TMPTMA PIC has shown good structural properties up to 350F.[6] However, exposure to aggressive agents such as water or brines at elevated temperature may drastically reduce the useful temperature range, as chemical attack is greatly accelerated. High temperature polymers of 90 percent diallylpthalate - 10 percent MMA and 70 percent MMA - 30 percent TMPTMA have been observed to deteriorate in exposure to brine at temperatures of 250F or more.[5,6] In a series of fire resistance tests with MMA PIC, it was found that at 500F the polymer was liquified and no longer contributed to the strength of the concrete. However, upon cooling to room temperature, the PIC specimens completely regained their strength. At higher temperatures, the polymer degrades and strengths are not recovered upon cooling. Surface burning characteristics tests on MMA, MMA-TMPTMA, and polyester-styrene PIC specimens in accordance with ASTM Designation E 84-70, based upon a scale of zero for asbestos-cement board and 100 for red oak floor, indicated zero fuel contribution and flame spread rates of 10 to 15 for all those PIC materials, and a smoke density factor of 15 for polyester-styrene PIC and zero for MMA-TMPTMA PIC.[46] It should be noted that there is no universal or absolute fire test, and test results may vary significantly depending upon type of test, test conditions, and characteristics of the sample, particularly if the sample may contain surface deposits of excess polymer. Some tests have shown smoking, odors, and fuel contribution from melted polymer.

10.11.6 Abrasion Resistance. Modest improvements in abrasion resistance have been reported for PIC.[1,3,38]

10.11.7 Toughness. The toughness of PIC should not be improved over that of conventional concrete and may be slightly decreased.

10.11.8 Fatigue Strength and Dynamic Conditions. The flexural strength of partially impregnated beams under cyclic loading was studied by Fowler, et al.,[38] using MMA-TMPTA. The results are shown in Table 10.14. Fatigue testing of MMA-impregnated concrete bars is continuing at BNL.[47] Specimens have undergone 100 million test cycles at 77 percent of the ultimate strength without failure. No information on the dynamic response of PIC could be located.

10.11.9 Permeability, The permeability of PIC should be less than that of conventional concrete due to the filling of the pore space in the concrete with a polymer. Table 10.12 shows typical data for absorption and permeability of PIC's.

10.12 USE OF PIC

Examples of the use of PIC in highway pavements could not be found. Considerable thought and effort have been devoted to the use of PIC in both new and existing deteriorated bridge decks, however,[15,20,25,38,48,49] and in prestressed PIC panels for bridge decking.[50]

10.12.1 In-Situ PIC Bridge Decks.

10.12.1.1 Sag Harbor, New York.[25] Two types of concrete distress existed on this bridge:

a. Badly deteriorated concrete.

b. Separation of the 4-in.-thick wearing course from the structural slab.

The PIC effort was designed to correct both these problems.

Several approaches to impregnation were tried. In the initial experiments, regions of deteriorated concrete were impregnated with MMA and a monomer mixture consisting of 75 wt percent styrene and 25 wt percent polyester resin (Plaskon 941). Undried and dried sections of concrete were treated with each monomer system. Infrared heaters, of the type commonly used with asphaltic concrete paving, were found acceptable for effective drying. Each unit could dry a 4- by 5-ft section of concrete in about 3 hours. Very limited success was achieved with the styrene/polyester mixture; probably due to either the high viscosity or the rapid curing time. Core samples of the MMA impregnated concrete indicated impregnation varying from 3/4 to 1-1/2 in. in depth.

Ponding MMA on a section of highly deteriorated concrete gave satisfactory results. Prior to polymerization, a PC overlay of 20 percent polyester-styrene and 80 percent crushed limestone was placed on top of the treated area. The monomers polymerized readily and traffic was restored in about two hours. Visual examination of the treated area one year later indicated no apparent deterioration.

A third method for successful impregnation consisted of injecting the monomer through a tube which was inserted into the deteriorated concrete to a point just above the sound structural deck. When a partial vacuum was applied to the surface of the deteriorated concrete during monomer addition, uniform and full impregnation was obtained.

Attempts to MMA-impregnate deteriorated concrete by first evacuating the surface, flooding the surface with monomer, and then applying an overpressure were not successful on deteriorated concrete, although it was suitable on relatively sound concrete.

Experiments were also conducted for impregnating the delaminating interface between the concrete overlay and the structural concrete sublayer. A series of holes was drilled through the overlay for injecting a mixture of 25 wt percent styrene and 75 wt percent polyester, and for venting air and overflow of the monomer mixture. Core samples which were taken about a week later indicated the presence of monomer, but that polymerization had not occurred. This was found to be due to dissolution of an asphaltic sealing membrane between the two concrete layers which inhibited the polymerization.

10.12.1.2 Denver, Colorado.[48,49] A field method for the surface impregnation of new bridge decks was demonstrated on a full bridge deck in the Denver, Colorado, area during October 1974. The bridge deck measured approximately 60 ft long by 28 ft wide and was treated in one application. The process consisted of drying the deck for 3 days, using gas-fired hot air heaters under an 18-in.-high insulated enclosure, cooling of the deck for 24 hr, impregnation of the deck with the monomer under a 3/8-in. blanket of dry, 50-mesh sand for 12 hr, and then *in situ* thermal polymerization under the insulated enclosure, again under the gas-fired heaters. The monomer used was 95 wt percent MMA- 5 wt percent TMPTMA with 0.5 wt percent azobis-dimethylvaleronitrile used as the catalyst. Cores taken from the deck indicated that a 1- to 1-1/2-in. surface layer of PIC was obtained. Previous research on laboratory test slabs indicated that the procedure results in a product having extremely high frost resistance and is virtually impermeable to water or salt solutions. Measurements of skid resistance, using a British portable pendulum tester, both prior to treatment and after treatment, indicated no loss of skid resistance due to the treatment.

10.12.1.3 Texas Bridge Deck Research. The use of a layer of sand as a monomer reservoir and as a means of reducing evaporation losses during the partial impregnation of bridge decks was developed at the University of Texas.[20,38] By placing a thin layer (about 1/4 in.) of dried fine aggregate over the concrete surface, it was found that depth of penetration, up to 1 in. (25 mm), could be achieved. An aggregate-polymer topping remained on the surface of the concrete after polymerization. Treatment of a test section consisted of the following steps:

a. Covering the specimen with polyethylene to prevent rain from wetting the surface and to dry the slabs.

b. Drying the surface for several days with an electric heating blanket.

c. Removing the heating blanket and covering the slab with about 1/4 in. of oven-dried lightweight fine aggregate.

d. Applying an initial 2000 to 3000 ml of the monomer system to each 12 sq ft of surface.

e. Covering the surface with polyethylene to retard evaporation.

f. Shading the surface to prevent a temperature increase which might initiate polymerization prematurely.

g. Periodically adding additional monomer to keep the sand moist for a minimum soak time of eight hours.

h. Applying heat to polymerize the monomer.

As of the writing of this report, plans had been made to construct two PIC bridge decks in Texas. One bridge, with an area of 5396 sq yd, was awarded for a bid of $10 per sq yd. The second bridge involved 966 sq yd and was awarded for a bid of $15 per sq yd.

10.12.1.4 Pennsylvania Test Track Bridge. Researchers at Lehigh University and the Pennsylvania State University are involved in selecting a monomer system and developing procedures for impregnating deteriorated concrete bridge decks, down to about 4 in. or below the level of the top layer of steel reinforcing. Following laboratory studies, the research team has successfully impregnated two 3- by 12-ft sections of the deck on the bridge on the Pennsylvania Test Track, near State College, Pennsylvania.[15]

10.12.2 Precast, Prestressed PIC Bridge Decking. The concept of precast, prestressed panels is reviewed in Chapter 13.

Lockman and Cowan[50] reported on a sequential bridge deck system consisting of precast, prestressed PIC panels. The major considerations in this laboratory study were if that the panels would resist chemical and freeze-thaw attack, be practical to use on a bridge, economical to produce, and meet the AASHTO specifications for HS-20-44 vehicle loading for bridge decks.

The structural design resulted in precast, prestressed panels which were 16 ft long by 4 ft wide by 6 in. thick with longitudinal tongue and groove joints. After impregnation, the prestressed concrete panels would be placed transversely across the supporting girders, and subsequently post-tensioned together in the longitudinal direction of the bridge. Special connections were designed to provide composite structural action between the panels and the girders and to provide a smooth riding surface. During the design process, it was recognized that total drying of the precast concrete, at "normal" temperatures could cause substantial loss in prestressing force, due not only to creep and shrinkage of the concrete, but also from relaxation of the prestressing strands. As a result, drying was conducted at only 200F, which took almost 2 weeks. Following drying, the panels were impregnated with MMA and 1 percent butylazo isobutyronitrile catalyst. Polymerization was achieved by flooding the chamber with 150F water and then raising the temperature to 160F for at least 3 hr.

Laboratory structural tests included static and fatigue tests on individual panels, groups of three panels, and miscellaneous special specimens. Tests were also conducted in PIC specimens at temperatures as low as -40F and on prestressing strand at temperatures as high as 350F to clearly define the stress-relaxation characteristic. The results of the tests indicate that the design is valid, with the ultimate strength and structural behavior of the prestressed PIC decking system exceeding design assumptions. A preliminary economic study indicates that the cost of this type of bridge deck may not be excessive, especially if long-term maintenance costs are considered. This same approach could be applied to precast panels for pavement slabs.

POLYMER-PORTLAND CEMENT CONCRETE

Polymer-portland cement concrete (PPCC) has been prepared with both pre-mix polymerized and post-mix polymerized materials. The pre-mix polymerized materials include latexes and polymer solutions or dispersions. The post-mix polymerized PPCC has been made with a number of resins and monomers.

10.13 POLYMER LATEXES

A latex consists of very small (0.05 to 1.0 μm diameter) spherical particles of polymers held in suspension by the use of surface-active agents. The polymer latex is usually formed directly by emulsion polymerization of the monomer and typically contains about 50 wt percent solids. Polymer latexes are generally copolymer systems of at least two or more monomers with a possible addition of a plasticizer and other modifiers. Examples of polymers used in latexes are poly(vinyl acetate) (PVAc), polystyrene, polyvinyl chloride, natural rubber, and polybutadiene. Some general characteristics of latexes are shown in Table 10.15.

Latex modified concretes are the oldest type of concrete composites which contain polymer and, at present, represent the large majority of commercial applications of polymer modified concrete in the United States. Their commercial success is most likely due to the fact that they can markedly improve the properties of ordinary concrete and mortar without requiring any significant changes in process technology.

The variability of the polymer latex formulations makes comparisons between PPCC's difficult. Many investigators have not reported the exact polymer formulations, probably because this information was not available to them in most cases. The importance of the emulsification agent incorporated in the latex during its manufacture has usually not been considered. Nevertheless, a number of suitable latex formulations have been developed for commercial applications and are finding increasing use because they greatly improve the shear bond, tensile, and flexural strength of cements and mortars compared to unmodified controls.

10.3.1 Thermoplastic Latexes. Polyvinyl esters[37,51-57] were among the first polymer types to be evaluated for use in PPCC, but are susceptible to rapid alkaline degradation, forming acetic acid, carboxylic acis, and polyvinyl alcohol.[51] The resulting PPCC, while possessing improved flexural and shear bond strength when dry, rapidly loses strength when wet because of the water solubility of polyvinyl alcohol. Polyacrylics[37,51,57] are superior to polyvinyl esters in this respect. However, slow alkaline degradation necessitates dry curing and limited exposure to moist conditions in practical applications. Both polyvinyl esters and polyacrylics can show improved performance by the use of suitable copolymer formulations.

Polyvinyl chlorides[37,51,55,57] have also been evaluated. A specially formulated polyvinylidene chloride-PVC copolymer (Saran) is commercially available. It produces PPCC of superior mechanical properties and excellent durability under moist conditions.[58]

Polystyrene[37,51,55,57] does not appear to have been commonly used in PPCC as a homopolymer. However, a thermoplastic styrene-butadiene latex formulation that affords improved durability and moisture resistance is commercially available.[58]

10.13.2 Elastomeric Latexes. Natural rubber latexes[37,51] were one of the first polymers used to modify portland cement systems. The relatively poor mechanical properties, however, have led to the development of PPCC synthetic elastomers, such as acrylonitrile-butadine,[37,51] neoprene,[37,51,57] and styrene-butadiene.[37,51,55,59,60,61] Synthetic rubber PPCC is more flexible, but has somewhat lower strength than thermoplastic PPCC.

10.14 POLYMER SOLUTIONS

Both thermosetting and thermoplastic water-soluble polymers of various kinds have been added to fresh concrete.[51,62] The thermosetting polymers include epoxies, amino resins, polyesters, and formaldehyde derivatives. The thermoplastics include polyvinyl alcohol and polyacrylamides. Work has been done in the USSR[63] with water-soluble polyorganosiloxanes to improve strength and water-tightness of concrete. Water soluble poly-electrolytes such as polyacrylamide, polyacrylonitrile, polymethacrylic acid, polyvinyl acetate-maleic anhydride, and polyisobutylene-maleic anhydride have been used to induce flocculation of the cement paste to improve rheological properties. Cellulose ethers and starch ethers have long been used in grouts, mortars, and concretes for improved water retention and cohesiveness during placing of the material.

10.15 MONOMERS

Attempts to prepare PPCC by adding vinyl monomers directly to the wet mix, followed by a subsequent polymerization during or after the curing of the portland cement, have in general not been very successful. Some problems have been encountered in uniformly dispersing the monomers throughout the mix, as the organic monomers are essentially insoluble in water. Furthermore, these organic compounds frequently interfere with the hydration of cement and may also suffer slow alkaline degradation. Thus adequate strength is not obtained after polymerization. Extensive alkaline degradation has been found with MMA, acrylamide, vinyl acetate and, to lesser extent, zinc acrylate. Acrylontrile and styrene appear to be alkaline resistant and give improvements over unmodified controls. Furfuryl alcohol PPCC has been reported as promising in the Soviet Union, but it is reported as requiring up to 45 days of curing to develop full strength.[37,63] This system has also been under limited study in the United States.[64]

10.16 THERMOSETTING RESINS

A polyester formulation has been used in England to produce "Estercrete."[65] A polymer/cement ratio of 0.3 or higher was used together with a water-soluble redox catalyst for the condensation reaction. When the system is mixed with water, the catalyst is activated, polymerization occurs, and cement hydrates. Such systems are very rapid setting, but with the high polymer loadings the system can be regarded as polyester filled with hydrated portland cement as well as hydrated portland cement paste filled with polyester.

The use of epoxy in concrete[5,37,51,66-71] has been the subject of more recent research and development in several countries. Water-soluble epoxy resins have been developed in the USSR,[37,51,68] while water-dispersible epoxy resins have been patented in Japan[66] and have recently been developed in this country.[69] The recommended loading of 10 to 20 percent by weight of cement and partial replacement of epoxy by other thermosetting resins, such as phenolics, is possible.

Polyurethanes have been investigated[72,73] for use in portland cement concrete and formulations have recently been patented but not yet developed commercially. Phenol-formaldehyde and urea-formaldehyde resins have been investigated by Russian workers,[37] primarily as plasticizers.

10.17 PPCC FABRICATION

The fabrication of PPCC is very similar to that of conventional portland cement concrete. Organic materials in either a powdery or dispersed form are added to the mixture during mixing. Because most organic polymers and their monomers are incompatible with a mixture of portland cement, water, and aggregate, a review of PPCC fabrication is in large measure an examination of how these basic incompatibilities are overcome. Extensive field experience has been gained with pre-mix polymerization (polymer latex). The problems anticipated for post-mix polymerization systems should be of a similar nature, however.

10.17.1 Latex Modified Concrete. The mixture proportions of latex PPCC will vary in much the same way as do those of normal concretes and mortars, depending on the end use application. Typical mixture porportions for latex modified mortars and concretes are shown in Table 10.16. The materials used in PPCC are the same materials used in regular concrete with the addition of the latexes themselves.

Any portland cement can be used in latex PPCC including modified cements such as white portland cement, waterproof portland cements, and shrinkage compensating cement (Chapter 8). However, in no case should air-entrained portland cement or air-entraining agents be used since polymer latexes already entrain considerable amounts of air. In fact, anti-foaming agents are generally recommended to control excessive air entrainment. Latexes are not very effective in lean concrete mixtures which contain 500 lb or less of portland cement per cubic yard of concrete. Normally, latexes are used in mixtures containing anywhere from 564 to 846 lb of portland cement per cubic yard of concrete. The major difference in the mixture proportions of latex concrete or mortar mixes in comparison to standard mortar and concrete mixes is that the water content of the mixture is reduced on a volume basis by the absolute volume of polymer that is added to the mixture. Polymer latexes generally act as water-reducing agents.

The same batching, mixing, and placing equipment used for conventional concrete can be used for PPCC. Good form release agents are necessary because latex modified cement systems adhere well to moist substrates including concrete, wood,steel, etc. Excellent results have been obtained by using silicone mold release agents, stearic acid, zinc stearate, or heavy applications of typical oil release agents. Care should be taken not to over vibrate PPCC, as the latex will migrate with the water to the surface resulting in a polymer layer on the top surface of the concrete and at the same time lowering the amount of polymer in the body of the concrete.

The optimum curing of latex PPCC is different than that of ordinary concrete. In normal concrete, maximum properties are obtained by moist curing the concrete for as long a period of time as possible up to 28 days. This is detrimental in latex portland cement systems. By the incorporation of latexes in these systems, a material is added which requires a different curing mechanism which is best achieved under the environment of less than 100 percent relative humidity. The optimum properties in latex-modified portland cement systems are obtained by moist curing the resulting concrete from one to three days followed by dry curing of the concrete at ambient conditions. The curing of latex PPCC may be accelerated by the use of heat. However, steam heat has been found to be detrimental to the gain in strength properties of these systems. The use of moist curing until after final set, followed by accelerated curing with dry heat, has proven to be an advantageous way of obtaining early strength properties.

10.17.2 Water-Soluble Polymer Concrete. As the name implies, these polymers are added to the mixture in the form of a water solution during the mixing operation. Normal mixture proportioning and batching procedures can be followed. The mechanism of combination of the water soluble poly-electrolytes and liquid resins with cement differs from that described for latexes in that most of these polymers are thermosetting. The exception is polyvinyl alcohol which is a thermoplastic.

Curing for the concrete usually involves some period of standard moist curing followed by curing at elevated temperatures. This can be achieved by hot air, hot water, or steam at either low or high pressures.

The cellulose ethers and starch ethers form solutions with water that are very viscous, and the viscosity persists when the solutions are used to hydrate hydraulic cements. Control of the viscosity is by choice of polymer and concentration with a range of 4 to 150 poises.[74] The polymer is added to the mixture while in solution and normal mixing procedures are followed. No special curing is necessary.

10.18 SAFETY CONSIDERATIONS

In general, latexes are not considered to be toxic materials. They are safe materials to handle and require only normal precautions in their use. In portland cement systems, the hazards of handling the portland cement itself are far more severe than those involved in the handling of the latexes. If latexes accidentally come in contact with the eyes, the eyes should be flushed with copious amounts of water for 15 minutes. It is then advisable to consult a physician. Repeated exposure of latex PPCC to the skin can cause mild irritation in much the same manner as conventional portland cement systems. Materials should be washed off the skin with water before they have set. Clothing contaminated with latex or latex polymer cement concrete should be washed prior to rewearing. In post-mix polymerization PPCC, the precautions discussed under Section 10.10 should be observed.

10.19 MECHANICAL PROPERTIES OF PPCC

In contrast to ordinary concretes, where modifying admixtures are used in very small amounts, PPCC contains large amounts of polymer which supplement the cement as a binding material. The presence of a distinct polymer phase confers desirable properties upon the concrete, such as enhanced ductility, improved durability, and superior adhesive properties. A comprehensive review of PPCC is currently in preparation at WES and should be available in late 1977. Other abbreviated views of PPCC have also been published.[37,75,76]

10.19.1 Pre-Mixed Polymerized PPCC. Pre-mixed polymerized PPCC is most generally prepared by adding a polymer emulsion, or latex, at the mixer. It is then commonly referred to as "latex modified concrete." Comparison of results obtained by different investigators is difficult, due not only to differences in mortar or concrete mix design and materials, but also to differences in the properties of the latex. Such factors as the size of dispersed particles and the type and amount of chemicals designed to act as emulsifying agents, to enhance the stability of the latex, or its coalescence on drying all have an effect on the subsequent performance of PPCC.[37] In general, only latexes that form a film when coalesced are of value. The nature of the polymers will affect the properties of the latex modified mortars to some extent as shown in Table 10.17, although differences are more pronounced with durability aspects. The polymer loading affects mechanical properties. Elastomers, such as styrene-butadiene generally give latex modified concretes with greater flexibility than other polymer latexes. Polymer types that have been investigated are mentioned in Section 10.14.

10.19.1.1 Freshly Mixed Concrete Properties. Polymer latexes will modify the behavior of fresh concrete in ways that may be beneficial or detrimental to strength. Many latexes will delay the setting of concrete and mortars. Generally, the latexes entrain excessive amounts of air (due most probably to the presence of surface active agents providing a stable latex) which must be controlled by the use of anti-foaming agents. These chemicals also increase the workability of the concrete, however, and allow reductions in water content at a given slump. Other admixtures such as calcium chloride, have been used successfully in PPCC.

10.19.1.2 Strength. A major effect of polymer latex is to improve the ductility of the cement paste or mortar. This is reflected in higher tensile and flexural strengths and increased strain at failure[58,77,78] (Figure 10.1 and Table 10.17). Most optimum properties are attained only when the latex modified concrete is dry-cured at 50 percent RH rather than continuously moist cured, as is required for conventional concrete. Although this curing regime limits the hydration of the cement, removal of water during dry curing allows the small particles of polymer in the latex to coalesce into a continuous polymer film around partially hydrated cement grains and aggregate particles. It is believed that a major benefit of the polymer vis-a-vis strength may be a strong cement-aggregate bond.[76,79] The polymer film may also, through its high tensile strength and elongation, effectively halt propagating microcracks and hold existing microcracks together.[79]

A serious disadvantage of some latex modified mortars formulations has been their tendency to lose strength when immersed in water or exposed to high humidities (Table 10.18). This is most probably caused by a partial reversion of the polymer phase to the dispersed state. The poor performance of PVAc in this regard is due in part to hydrolysis of the polymer.[37,58] Generally, redrying will allow strengths to be regained if permanent chemical change has not occurred. This problem, which initially was severe enough to limit latex modified mortars to interior applications, has now been largely overcome by the use of new co-polymer formulations, and latex modified cements with excellent water resistance are now being used commercially.

10.19.1.3 <u>Dimensional Changes</u>. Shrinkage of latex modified mortar specimens during dry curing has been observed, with its magnitude being dependent on polymer type and loading. Results are somewhat varied for different investigations. Polyvinyl acetate and acrylic latex modified mortars have high shrinkage compared to control.[75,79] Shrinkage of styrene-butadiene concretes have been variously reported as much lower,[59,75] or slightly lower[58,75] than controls. Vinyl chloride-vinylidene co-polymers may have shrinkages higher or lower than controls.[58,75] Creep of latex modified cement has not been studied extensively. In studies reported by Solomatov[37] on latex modified concrete containing vinyl acetate-dibutyl maleate co-polymer, the latex modified concrete showed much higher creep in bending than plain concrete. The creep curves showed very rapid initial creep due to the "delayed" elasticity of the polymer. At 50C the specimens failed under a stress equal to 0.33 of the ultimate static strength, since the glass transition temperature of the polymer was exceeded.

10.19.1.4 Durability. An important aspect of latex modified concretes is their improved durability over conventional concrete. This is due partly to reduced porosity as a result of lower w/c ratios and partial filling of pores by polymer, but existing pores also tend to be sealed by a continuous polymer film.[42] Furthermore, existing micro-cracks tend to be held together by microfibers of polymer and so will not become a point of entry for damaging chemicals. These features are reflected in reduced permeability, water absorption (Figure 10.2) and water vapor transmission.[37,58]

The resistance of PPCC's to chemicals depends on the nature and amount of polymer added and the nature of the chemical. Thus, polyvinyl acetate modified concretes do not resist acids and alkalis very well but are stable toward organic solvents such as mineral oils. In contrast, elastomers such as styrene-butadiene are resistant to acids and alkalis but will be attacked by some organic solvents (Table 10.19). Saran latexes afford resistance to acids and alkalis as well as to most organic solvents.[58]

Latex modified concrete materials generally have excellent frost resistance since they contain air entrained by the latex. Even under water, resistance to freezing and thawing is maintained despite loss of strength.

10.19.1.5 Abrasion Resistance. A marked improvement is found in the wear resistance of latex modified concrete. Abrasion and impact resistance are considerably increased over plain concrete (Table 10.20).

10.19.1.6 Toughness. The ductility of latex modified mortars is significantly improved over that of conventional concretes and thus should result in improved toughness of the material.

10.19.1.7 Adhesion. All latex modified concrete materials show excellent adhesive properties, a characteristic very valuable in potential applications as overlays and toppings. The shear bond strengths for dry-cured mortars given in Table 10.17 exceed those of plain concrete; hence, when latex modified material is used to patch deteriorated concrete, the shear failure plane should go through the concrete rather than through the interface.

10.19.1.8 Temperature Resistance. Concretes containing thermoplastic polymers will rapidly lose mechanical properties at higher temperatures when their glass transition temperatures are reached. For example, a polyvinyl acetate co-polymer latex modified mortar lost 50 percent of its strength and dynamic modulus at 45C[37] compared to that at 20C. Increased creep has been mentioned in Section 10.19.1.3. The thermal coefficients of expansion are similar to conventional concrete.

10.19.1.9 Fatigue Strength and Dynamic Conditions. No information on the fatigue strength or dynamic response of latex modified concretes could be found.

10.19.1.10 Permeability. The permeability of latex modified concrete has been reported as being both improved and not improved. This is probably related to the polymer in use and whether or not it reverts to a dispersed phase when in contact with water.

10.19.2 Post-Mix Polymerized PPCC. This approach to the preparation of PPCC consists of integrally mixing monomers or prepolymers (e.g., vinyl monomers, epoxy resin prepolymers), portland cement, water, and aggregate. The polymerization process takes place during and/or after the hardening of the concrete. Initial laboratory studies have been disappointing and have precluded a thorough investigation of these materials. Some of the problems encountered which contribute to poor performance are (1) interference with hydration of the portland cement; (2) chemical reaction between monomer and the cement paste, e.g., hydrolysis; (3) difficulty in dispersing the organic component through the mix; and (4) poor polymer-aggregate bond. Information on post-mix polymerized PPCC is scarce.

10.19.2.1 Vinyl Monomers. The effects of vinyl monomer additions on compressive strength are shown in Table 10.21.[1,4,45,80] It can be seen that considerable differences are formed in the extent to which post-mix polymerization affects the strength of concrete. The general conclusion is that improvements in strength are usually small and may depend on the quality of the polymer used. Considerable decreases in strength may be observed which can be traced to one or more of the problems listed above. In some studies, MMA and vinyl acetate failed to develop strength, as did other monomers such as acrylamide and styrene-TMPTMA.

10.19.2.2 Styrene Monomers. Results of a single study[45] on mortar specimens have shown improved durability for post-mix polymerized polystyrene-PPCC. Exposure to hot water, hot seawater, 10 percent HCl, and 0.5 M Na_2SO_4 were investigated, and performance compared favorably with PIC.

10.19.2.3 Epoxy Resins. Catalyzed epoxy resin was added to fresh mortar as a partial replacement for portland cement.[5,71] The results are shown in Figure 10.3. It can be seen that at least a 50 percent weight replacement of cement by epoxy is required to give a significant improvement in strength. Further, adequate high temperature curing is needed to realize the full potential of the system. These results suggest that the system could be regarded equally well as a filled epoxy rather than a modified mortar. The effect of adding fly ash indicates that the properties of the filler are important, however.

Water soluble epoxies[37] and epoxy resin emulsions[75] have been used successfully for improved concretes and mortars. A major advantage of these materials is their ability to be successfully cured under moist conditions. Improved flexural strength and durability are obtained.

10.19.2.4 Polyester-Styrene Resin. Nutt[65] describes a product called "Estercrete" where cement is partially replaced by a polyester-styrene resin. Polymerization is initiated by the dissolution of a redox catalyst in the mixing water. The product has the advantage of rapid setting and strength development and these characteristics can be varied considerably. The material has good wear resistance and is designed for rapid repairs of pavements and slabs or as toppings and overlays.

19.19.2.5 Other Resins. Solomatov[37] reports on the use of water soluble ureaformaldehyde resins to provide stronger, more durable concrete. Furfuryl alcohol-aniline polymer provides a concrete highly resistant to gasoline and oil.[37] Prolonged curing in the moist state is required to prevent cracking and attainment of adequate strength.

10.20 USE OF PPCC

There are several PPCC materials which have been developed by various commercial companies for use in patching and overlaying deteriorated bridge decks.[81] Latex modified concrete, because of its excellent adhesion to base concrete, its improved tensile and flexural strength, its excellent freeze-thaw resistance, its resistance to penetration of chloride ions and its ease of application has proved particularly successful. Since 1957, hundreds of bridge decks in the United States have been restored with PPCC.[82,83,84] It is important to note that the successful use of such materials has been shown to be dependent upon the following: (1) removal of asphaltic wearing or sealing layers; (2) complete removal of deteriorated and/or loose concrete; and (3) complete cleaning of the exposed surface of the underlying concrete bridge deck.

Although PPCC has probably been used in pavements on grade, no written descriptions of such applictions were located for this review.

POLYMER CONCRETE

Polymer concrete is a composite material consisting of a polymer matrix and particulate fillers, prepared by the integral mixing of a polymerizable material (such as monomer or resin) and aggregate. Polymerization is usually obtained through a catalyst-promoter system without the introduction of radiation or thermal energy. Various polyesters, epoxies, furans, and PMMA have been used as the matrix of PC because of the reasonable compromise between relative ease of polymerization and desirable properties.

10.21 MONOMERS AND RESINS

Most of the work on PC has been chiefly with polyester-styrene resin systems, and to a lesser extent with furan and epoxy and vinyl ester resin systems.[37,53,85,86,87] The polyester resins are attractive because of moderate cost, availability of a great variety of formulations, and moderately good PC properties. The furan resins have been investigated in Europe,[37,53,87] and are low cost and highly resistant to chemical attack. The epoxy resins are generally somewhat higher in cost, but may offer some advantages such as adhesion to wet surfaces with specially formulated epoxies.

Low viscosity monomers, such as MMA and styrene, have been also investigated. They are easily mixed with aggregates, giving a mix which can be readily compacted into a dense PC of low porosity and relatively low polymer content. The low viscosity monomers also penetrate fractures and voids in the aggregate particles. PC made with an MMA-TMPTMA monomer system appears to have structural properties comparable to PIC and superior durability.[6]

10.22 ADDITIVES AND MODIFIERS

10.22.1 Plasticizers. PC has a greater ductility than either PIC or conventional concrete. Nevertheless, efforts have been made to further increase the ductility of PC through the addition of a plasticizer to the monomer.[6,88] An example is the co-polymerization of methyl methacrylate with butyl acrylate as a plasticizer.[6]

10.22.2 Flame Retardants. Since a relatively small amount of polymer is actually present in PC, the composite is nonburning or self-extinguishing by standard flamability tests. It is probable that the loss of structural integrity in PC exposed to fire conditions may be more important than questions concerning the flamability of the material. PC appears to be essentially self-charring and flame retardants probably are not necessary, but nevertheless can be used as in the case of PIC.

10.22.3 Silane Coupling Agents. One of the most beneficial modifications to PC is the addition of silane coupling agents to the monomer system. The coupling agent serves to increase the interfacial bond between polymer and aggregate and hence increase the strength of the composite. Several techniques have been used for applying silane coupling agents to PC composites.[5,6] The most practical technique is to add the compound directly to the monomer system prior to mixing, but the greatest strength is obtained when the aggregate is pretreated with the coupling agent.

10.23 PC POLYMERIZATION

Most of the monomer and resin systems for PC are polymerized at room temperatures. The vinyl monomer systems can be polymerized with a variety of catalysts, but are most commonly polymerized using benzoyl peroxide with an amine promoter. The polyester-styrene systems likewise are polymerized with a similar variety of promoter-catalyst systems. The most common system is methylethyl ketone peroxide with cobalt napthanate promoter. These systems polymerize satisfactorily at room temperatures and the application of heat is not essential. A final heat treatment may be used, however, to ensure that the highest degree of conversion is obtained. Other room temperature systems include amine curing agents for epoxy resins.

Gamma radiation from a cobalt-60 source also can be used for polymerization, as in PIC, but this method is less likely to find favor because of the availability and initial cost of radiation sources and the hazards involved in field applications.

10.24 PC FABRICATION

The batching, mixing, and placing techniques for producing PC are largely based on adaptation of existing equipment and methods for producing portland cement concrete. A knowledge of polymer chemistry is helpful but not absolutely essential, as directions for curing mixes are readily available from the resin manufacturers and from the published literature.

10.24.1 Monomer and Resin Systems. Most of the work on PC has been with epoxy, polyester, and furan resins and more recently with MMA and styrene monomer systems. However, it is conceivable that many other resins could be used as well as many polymers through the use of solvent or fusion molding techniques. The important considerations in choice of the resin system include low cost, durability under anticipated exposure conditinos, adhesion to aggregate, handling properties, and ease of curing.

10.24.2 Mixture Proportions. The design of a PC mix to optimize the properties of the resultant material is largely accomplished by aggregate gradation to give a void volume which will require minimal amounts of monomer or resin to fill the voids and to give good workability.[5,6,37,53,86,87,89] In general, the aggregate should be dried to less than 2 percent moisture prior to use, although some epoxy resins are less affected by moisture and, therefore, the drying conditions are less stringent. The major mix variables are maximum particle size, gradation, and composition. Crushed stone and natural sand and gravel aggregates are generally used, but finely divided materials such as portland cement, powdered chalk, clay, fly ash, and silica flour, have also been incorporated as fillers and to improve workability of the mix. Test results indicate that aggregate type and composition do not significantly influence the strength properties of the mix[89] but do affect the durability.[32] Aggregate gradation and maximum particle size influence the amount of resin required to coat the filler particles and to fill the voids. Tests have indicated that for a well-graded aggregate, larger maximum particle sizes require less resin; also gap grading tends to reduce the amount of resin required. Conversely, smaller maximum particle sizes produce higher strength mixes. An aggregate size distribution developed at the USBR,[6] which when mixed with 7 to 8 percent MMA, produced specimens with compressive strengths of ~ 19,000 psi is given in Table 10.22. Dense binder mixes of the type used in asphalt concrete will produce composites with a strength of 13,000 psi when mixed with 9 percent MMA.[25]

The monomer content of the mix, a dependent variable, is the minimum necessary to coat the aggregate and to fill the voids. Excess monomer will bleed to the surface due to the low density relative to the density of the aggregate. Monomer concentrations ranging from 5 to 30 weight percent, depending on void volumes, have been reported. The lowest loadings were obtained in pipe when a compaction method which involved pressure, vibration, and centrifugal force was used in conjunction with a graded aggregate and filler.[32]

10.24.3 Fabrication Materials. Conventional mixing equipment may be used for PC. The use of some resin systems, such as polyesters and epoxies, may present some cleaning problems, which ordinarily can be handled by solvents. These problems do not exist with MMA, which is an excellent solvent and will tend to evaporate before it will polymerize in the mixer. However, volatile and potentially explosive monomers, such as MMA, will require nonsparking and explosion proof equipment. Mixing should be done in a closed system or outdoors in a well ventilated area. Some of the chemicals may be irritants or toxic and should be handled in accordance with recognized safe practices.

The PC mix is cast into forms or molds in a manner similar to conventional concrete. Wood, steel, glass, and paper molds have all been used successfully. A great variety of mold-releasing agents have also been used, such as silicone gels, vegetable oils, automobile wax, and paraffins. After placing, the mix should be consolidated by external vibration, rodding, mechanical pressure, or application of vacuum to remove entrapped air. The harshness of some mixes may prohibit the use of internal vibrators, in some cases, but in other cases with well graded mixes internal vibrators work very well.

10.24.4 Curing. Curing of the PC may be performed by radiation, thermal-catalytic, or catalyst-promoter methods. (See Section 10.9.) Catalysts and promoters are added to the monomer prior to mixing with the aggregate. Curing tmes may be varied between a few minutes and several hours. Full strength is attained when polymerization is completed.

10.25 SAFETY CONSIDERATIONS

Although the monomers are toxic and flammable, the use of well established safety procedures allows them to be used without undue difficulty. The use of explosion proof equipment is discussed in Section 10.24.3.

10.26 MECHANICAL PROPERTIES

Polymer concretes differ from portland cement concretes, PIC, and PPCC in that only organic polymer materials are utilized as the binder or matrix of the concrete. Sulfur is an inorganic polymer and can also be used as a binder for concrete. This material is treated in Chapter 6 which is concerned with noncalcareous inorganic cements.

The properties of PC are largely dependent upon the properties of the polymer binder and the amount of polymer in PC, modified somewhat according to the effects of the aggregate and filler materials, and accordingly, would be expected to differ somewhat from the properties of the other types of concretes. Many engineering properties of PC have been investigated in Europe since 1957 and also in Japan, and there have been several field applications of PC.[37,87,90] Studies of PC in the United States have been of a more limited feasibility nature and have been usually directed toward determining only compressive strength and deformation characteristics.[4,5,89,91-94]

10.26.1 Strength. Most strength and deformation data for PC have been determined from strength tests of short duration and creep tests at stress levels below the long-term strength. The resulting stress-strain relationships do not reflect the magnitude of the viscous deformations which might occur under service loadings, and the reduced strength associated with extended durations of loading. Further, the effects of environments other than laboratory conditions on the response of PC to mechanical stress have not been studied in detail.

Representative properties of polymer concretes[37] based on tests of short duration are presented in Table 10.23 for comparison with properties of portland cement concretes. Strengths will tend to decrease, and actual deformations will tend to be larger than those computed from the modulus of elasticity when the duration of loading exceeds the duration of laboratory tests. The effects of temperature on properties of MMA-TMPTMA PC are shown in Table 10.24.[6]

Mechanical properties are dependent on the relative volumes of any given polymer matrix material and fillers. Modulus of elasticity, compressive and flexural strength, and density tend to increase with increased filler content until compaction becomes difficult, then these properties tend to decrease with further increase in aggregate content.[87] Improved matrix-aggregate bonding tends to increase the strength of PC.[5]

10.26.2 Dimensional Changes. Thermosetting polymers, such as polyester and epoxy, exhibit shrinkage during hardening. The shrinkage associated with hardening of the matrix is reduced by increasing the amount of aggregate filler as indicated in Figure 10.4 for polymer concretes, which utilized polyester, furan, and epoxy.[87]

Thermosetting polymers have thermal expansion coefficients which are high relative to portland cement concrete. The thermal expansion coefficient decreases with the addition of aggregate filler, and the thermal expansion coefficient of PC is on the order of 20 to 30 x $10^{-6}/^{o}C$.[87]

The response of PC to mechanical stress is strongly influenced by the viscoelastic behavior of the polymeric matrix material. The response of amorphous polymers (usually used in PC) to mechanical stress is complex, being both temperature-dependent and time-dependent, as well as being a function of molecular structure.[93] The response to stress varies somewhat among different types of polymers, such as PMMA or polyester, according to the chemical structure and molecular architecture of the polymer chains. A given type may also show some variations in response to stress as a result of large differences in the length of the polymer chains, process conditions, and thermal history.

Figure 10.5 shows the idealized response of PC in terms of creep strain vs. time for several different levels of axial compressive stress. It should be noted that temperature and humidity of the environment affect creep greatly.[37] Creep increases rapidly with an increase in temperature. Increases in creep of several hundred percent have been reported[91,94] within the range of anticipated summer temperature fluctuations (65 to 120F).

10.26.3 Durability.

10.26.3.1 Freeze-Thaw Resistance. This characteristic of PC has generally not been measured and no conclusions on this characteristic could be reached.

20.36.4.3 Chemical Resistance. Polymer concretes have high chemical resistance compared to portland cement concretes as indicated in Table 10.25.[37] These high resistances are dependent on complete polymer curing and on the use of chemical resistant aggregates.

10.26.3.3 Temperature and Fire Resistance. As noted in Section 10.22.2 the composite PC is generally nonburning or self-extinguishing by standard flamability tests. It will undergo loss of structural integrity and strength when exposed to elevated temperatures.

10.26.4 Abrasion Resistance. Specific information on the abrasion resistance of PC in pavements was not located, however, many applications of PC to industrial floors have indicated that with the proper selection of a durable, tough aggregate, its abrasion resistance is improved over that of conventional concretes.

10.26.5 Toughness. There is a wide variability in the toughness of PC composites. The influence of polymer properties on stress-strain relationships is shown in Figure 10.6.[6] PC made with MMA shows a nearly linear stress-strain relationship, high ultimate strength, and fairly abrupt failure. The addition of butyl acrylate increases the plastic behavior properties and produces a more ductile material. The ductile behavior is shown with the 80-20 MMA-BA comonomer system. Hence, the toughness will be dependent on polymer constuituents among other things.

10.26.6 Fatigue Strength and Dynamic Conditions. The dependence of the response of PC to the stress rate is indicated in Figure 10.7. Idealized stress-strain relationships under uniaxial compression are given for very slow ($d\sigma/dt \rightarrow \infty$) and very rapid ($d\sigma/dt \rightarrow 0$) stress rates. Under slow loading rates of relatively long duration, the response is governed by successive stages of elastic, viscoelastic, and viscous flow behavior of the polymer. The PC characteristically shows an initial linear stress-strain relationship, gradual departure from linearity as the material passes from viscoelastic to viscous flow behavior stages, and nonlinearity as the polymer undergoes viscous deformation, microfracturing and ultimate failure. Under very rapid or impact loading, the viscous deformations of the polymeric matrix material approach zero and the stress-strain relationship is nearly linear until the initiation of microcracking, which is associated with stress concentrations caused by the mismatch of matrix and aggregate mechanical properties.

No information on fatigue resistance of PC could be located.

10.26.7 Permeability. The permeability of a properly proportioned and made PC approaches zero and hence is greatly improved over that of conventional concrete.

10.27 USE OF PC

The use of PC in pavements has mostly been for repair work. In bridge decks, it is used for filling potholes[21] and for overlying deteriorated decks. A bridge deck in New York, New York, was repaired by filling a 3-ft- by 10-ft- by 18-in.-deep hole with PC.[25] The repair was on the Third and Lincoln Avenues bridge deck of the Major Deegan Expressway in the lower Bronx. The high traffic densities in this area prevent closing of a bridge lane for more than five hours. Since conventional repair methods could not be used, the hole had been covered with a steel plate. One lane was closed while the form work was prepared. The hole was filled 2 days later with a PC consisting of 13 percent monomer (95 wt percent MMA - 5 wt percent TMPTMA) and 87 percent aggregate. Due to an ambient temperature of about 50F, 2 percent benzoyl peroxide was used as the catalyst and a mixture of 2 percent dimethyl aniline and 1-2 percent dimethyl-p-toluidine as the promoter. A silane coupling agent was added at a concentration of 1.5 percent to enhance bonding to the adjacent concrete. Mixing and placing were completed in one hour, using conventional equipment. Polymerization was complete within one hour at which time the side forms were removed. Estimates of the compressive strength using a Windsor Probe indicated values ranging between 5400 and 8000 psi. The measured compressive strength of a cylinder cast from the same mix was 12,200 psi and the water absorption was 0.4 percent. At 3:00 pm, five hours after starting the work and two hours after completion of the placement, the lane was opened to traffic. Inspection of the top and underside of the section after six weeks in service indicated no apparent changes.

Kampf[95] reported on uses of epoxies as surface treatment, adhesive and binder on three jobs in the New York City area in 1964. In each case, the concrete in the bridge deck failed because of deflections of the bridge structure. The repairs met with varied degrees of success.

Polyester- and epoxy resin-based mixes can be used for repairing wide cracks or spalled areas in concrete roads or airfields.[74] Nutt[96] reported that heavy duty repairs for pavements, such as broken slab corners or whole bay failures, can be repaired with concrete containing up to 1-in. maximum size aggregate bound with a resin-portland cement filler composite when a special water-initiated polyester resin-cement system is used.

In conversations the author had with manufacturers of resins for PC, the general theme was that PC's were being used routinely throughout the United States for patching and repair in pavements but because of the small, limited effort in each repair, they generally are not documented for referencing purposes.

SUMMARY OF PERFORMANCE

10.28 EASE OF CONSTRUCTION

Of the three types of concrete containing polymers, the PPCC appears to be the simplest to produce and place followed in order of difficulty by PC and PIC. With the exception of taking special precautions with bond breakers, PPCC can be made and placed as simply as conventional concrete using the same equipment. PC often needs special batching and mixing equipment and always requires a technique to cause polymerization. PIC requires the actual construction of the concrete pavement followed by the drying of the pavement. It is then impregnated by various means with a monomer and then polymerized. The entire operation is quite time consuming but has been successfully done in bridge decks and in pavement-like surfaces in a dam spillway.[97]

10.29 EASE OF MAINTENANCE

Pavement made of PIC, PC, or PPCC should be no more problem to repair than a conventional concrete pavement except that in the PIC and PC pavements, special equipment may be necessary to manufacture the concrete so it conforms exactly to that of the pavement being repaired.

10.30 ENVIRONMENTAL COMPATIBILITY

There is insufficient information available to determine whether PIC, PC, or PPC pavements would be compatible with the environment. Other than the possible liberation of toxic fumes, which would be small in amount and localized, the limited data available suggest that polymers in concrete should not be a source of environmental problems. Further study is warranted, however.

10.31 AVAILABILITY AND COST

Availability of polymers and the additives and modifiers for use in concrete generally should not be a problem, although fluctuations in the petro-chemical industry in the past have resulted in temporary shortages. The cost will also vary depending on availability and market fluctuations and can be expected to continue to increase with time. All of the polymers are expensive and increase the cost of concrete significantly even when used in small amounts.

REFERENCES

CHAPTER 10

POLYMERS IN CONCRETE

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Table 10.1

Physical Properties of Common Monomers Used in PIC and PC

Monomer	Viscosity (centipoise)	Density gm/cc	Vapor Pressure (mm Hg)	B.p. (C)	Solubility in Water (%)
Acrylonitrile	0.34[c]	0.81	85[b]	77	7.4[c]
Diallyl phthalate	12.0[b]	--	2.54[f]	300	Insol
Methyl methacrylate	0.57[c]	0.94	35[b]	100	1.5[d]
Monochlorostyrene	1.04[c]	1.11	0.68[b]	180	0.0064[c]
Styrene	0.76[b]	0.91	2.9[b]	135	0.070[c]
Tert-butylstyrene	1.46[c]	0.88	1.0[e]	218	0.0005[c]
Vinyl acetate	0.43[b]	0.93	115[c]	73	2.5[b]
Vinyl chloride	0.28[a]	0.91	1660[b]	-13.9	Slight
Vinylidene chloride	--	1.21	599[c]	32	Insol

(a) -20°C (b) 20°C (c) 25°C (d) 30°C (e) 46°C (f) 150°C

Table 10.2

Properties of Some Polymers Used in PIC, PPCC and PC

	PS	PMMA	PVAc	PVC
Physical Properties				
Specific gravity	1.05	1.18	1.19	1.38
Glass transition temp. (°C)	93	100	70	80
Decomposition temp. (°C)	250	260	200	110
Water absorption (%)	0.03-0.05	0.3-0.4	3-6	--
Mechanical Properties				
Compressive strength (ksi)	11.5-16	11-19	--	10
Tensile strength (ksi)	5-12	8-11	2-4	
Flexural strength (ksi)	8.7-14	12-17	--	--
Modulus of elasticity (ksi)	400-500	400-500	40	30-60
Poisson's ratio	0.33	0.33	--	0.38
Impact strength (ft lb/in. of width)^2	0.25-0.4	0.4-0.5	--	--

Table 10.3

Glass Transition Temperature (Tg) of Co-Polymers

Co-Polymers	Tg, C	Co-Polymer	Tg, C
100% MMA	100	100% PS	93
90% MMA-10% DAP	185	60% PS-40% TMPTMA	213
70% MMA-30% TMPTMA	160	60% PS-40% poly (acrylonitrile)	110

Table 10.4

PIC Composition and Method of Curing

Concrete composition[a]	Method of Curing	Polymer Loading, wt %[b]	Compressive Strength, psi[c]
Standard (CP) mix, 6% air	fog[d]	7.2	24,750
CP mix, 2% air	fog	5.9	24,200
	HPS[e]	7.2	24,800
CP mix, 10% air	fog	9.5	23,400
	HPS	10.0	22,000
Low strength, 2000 psi	fog	7.2	25,000
	HPS	8.3	27,260
High strength, 10,000 psi	fog	6.1	25,300
	HPS	7.1	29,900
CP mix with expanded shale aggregate	fog	25.5	24,300
	HPS	29.8	26,000
CP mix with pit run Clear Creek aggregate	fog	10.0	21,750
	HPS	11.9	19,950
CP mix with high porosity limestone aggregate	fog	18.1	16,700
	HPS	20.2	17,150
CP mix with low porosity limestone aggregate	fog	6.9	22,400
	HPS	8.4	22,950

(a) All mixes are variations of standard CP-type concrete (See Ref. 1).
(b) Monomer, MMA; radiation-induced polymerization under water. All specimens are 3×6-in. cylinders oven-dried at 230°F prior to impregnation.
(c) Average of 8 specimens.
(d) Fog curing, 28 days.
(e) High-pressure steam curing at 250°F for 3 hr.

Table 10.5

Typical Mechanical Properties of PIC[6]

Polymer	Polymer Loading, wt %	Strength (psi)			Modulus 10^6 psi	
		Compressive	Tensile	Flexural	Elastic	Flexural
Unimpregnated	0	4,950	335	630	2.7	3.0
MMA	4.6-6.7	20,250	1,630	2,640	6.3	6.2
MMA + 10% TMPTMA	5.5-7.6	21,590	1,510	2,220	6.1	6.1
Styrene	4.2-6.0	14,140	1,100	2,300	6.3	6.3
Acrylonitrile	3.2-6.0	14,140	1,040	1,470	5.9	4.5
Chlorostyrene	4.9-6.9	16,090	1,120	2,380	5.6	6.3
10% Polyester + 90% styrene	6.3-7.4	20,500	1,500	3,300	6.5	6.4
Vinyl chloride[a]	3.0-5.0	10,240	675	--	4.2	--
Vinylidene chloride[a]	1.5-2.8	6,650	370	--	3.0	--
t-butyl styrene[a]	5.3-6.0	18,150	1,445	--	6.4	--
60% styrene + 40% MPTMA[a]	5.9-7.3	17,140	910	--	6.3	--

Note: Concrete dried at 221°F overnight and was radiation polymerized.
(a) Dried at 302°F overnight.

Table 10.6

Mechanical Properties of PIC Using Methacrylate (MA) Esters[38]

Polymer	Viscosity of Monomer (centistokes)	Loading Weight %	Compressive Strength psi	Modulus of Elasticity 10^6 psi
Control	---	0	6,610	3.5-4.5
MMA	0.60	5.27	16,350[a]	6.12
Isobutyl-MA	0.98	4.99	16,290[a]	5.42
Stearyl-MA	10.5	2.52	6,920	3.48
Isobornyl-MA	6.0	2.80	9,060	4.70
Isodecyl-MA	3.3	3.51	7,000	3.38
+20% MA	---	3.63	8,510	3.78
+40% MMA	---	4.97	14,060	4.85
+80% MMA	---	5.19	16,593[a]	5.90

(a) Exceeded capacity of the testing machine.

Table 10.7

Effect of Polymerization Methods on Strength of PIC[4]

Polymer	Strength, (psi)					
	Compressive		Tensile		Flexural	
	R.[a]	T.[a]	R.	T.	R.	T.
Control	4,950	5,260	335	388	632	666
MMA	20,620	18,160	1,630	1,510	2,640	2,290
MMA + 10% TMPTMA	21,950	19,000	1,510	1,250	2,200	---
Styrene	14,140	8,790	1,100	720	2,300	1,060
Chlorostyrene	16,090	14,390	1,120	1,200	2,380	1,580
Acrylonitrile	14,410	10,750	1.040	870	1,470	620

(a) R = Radiation; T = Thermal-Catalytic.

Table 10.8

Compressive Strength of PIC at Various Temperatures[4,5,6]

	Compressive Strength, psi				
	-23°C	21°C	121°C	143°C	177°C
Unimpregnated concrete	5,900	5,000	5,200	5,100	5,000
(60-40) Styrene-TMPTMA PIC	18,900	16,900	16,100	15,000	14,600

Table 10.9

Compressive and Tensile Creep of PIC

Loading	Days Under Load	Creep, millionths/psi	
		Unimpregnated Concrete	MMA PIC
Compression			
690 psi	836	0.40	0.09
800 psi	830	0.24[a]	(0.08)[a]
2313 psi	836	0.43	0.06
7000 psi	836	--	0.05
Tensile			
178 psi	836	0.16	(0.51)
345 psi	836	--	(0.24)

Note: 4-1/2 × 12-inch specimens, except as designated by [a].
Tests conducted at 73°F at 50 percent relative humidity.
Values in parenthesis () indicate negative creep; i.e., creep in the direction opposite of the loading.
(a) 6 × 12-inch specimens.

Table 10.10

Comparison of Creep of PIC Oven Dried Concrete and Undried Conventional Concrete

Specimen	Creep Deformation, millionths/psi Days Under 800 psi Compressive Load			
	30	360	1000	1500
MMA-PIC	(0.069)	(0.094)	(0.083)	(0.089)
Oven dried concrete[a]	0.058	0.089	0.095	0.096
Undried concrete	0.119	0.230	0.261	0.291

Note: 6- by 12-inch cylinders.
() indicate negative creep, i.e., an increase in the length of the specimens. Tests conducted at 73°F at 50 percent relative humidity.
(a) Unimpregnated concrete specimens, oven dried at 221°F prior to impregnation.

Table 10.11

Effect of Temperature on Creep of PIC

Loading Conditions	Test Temperature Degrees, F	Compressive Load psi	Days Under Load	Creep, Millionths/psi	
				Unimpregnated Concrete	Styrene-TMPMA PIC
Normal Strength Concrete					
Loaded at 250 F	250	800	62	0.24	---
		800	348	0.39	---
		2,313	62	0.20	0.07
		2,313	348	0.29	0.11
		7,000	62	---	0.08
		7,000	348	---	0.11
Loaded at 290 F	290	800	62	0.42	---
		800	348	0.60	---
		2,313	62	0.36	0.13
		2,313	348	0.48	0.19
		7,000	62	---	0.14
		7,000	348	---	0.18
High Strength Concrete					
Loaded at 290 F	290	2,313	30	0.30	---
		2,313	348	0.39	---
		2,313	889	0.41	---
		7,000	30	---	0.03
		7,000	348	---	0.04
		7,000	889	---	0.04
Loaded at 76 F	290	2,313	30	0.74	---
		2,313	348	0.78	---
		2,313	889	0.81	---
		7,000	30	---	0.07
		7,000	348	---	0.08
		7,000	889	---	0.09

Note: 4-1/2 by 12-inch cylinders.

Table 10.12

Permeability and Absorption of PIC

(Thermal-Catalytically Cured; Dried at 105C Prior to Impregnation)[3,4]

Property	Control		MMA	Styrene	MMA + 10% TMPTMA	Acrylonitrile	Chlorostyrene
	Undried	Dried					
Water absorption %	6.4	6.2	0.34	0.70	0.21	5.68	1.97
Water permeability (10^{-4} ft/yr)	5.3	29	1.4	1.5	1.2	---	---

Table 10.13

Durability of PIC[4]

Property	Control		MMA	Styrene	MMA + 10% TMPTMA	Acrylonitrile	Chlorostyrene
	Undried	Dried					
Freeze-thaw							
No. of cycles	740	440	3,650	5,440	4,660	4,120	1,800
% wt loss	25	28	2	21	0	6	10
Sulfate attack							
No. of days	480	605	720	690	630	540	300
% expansion	0.466	0.522	0.006	0.030	0.003	0.032	0.009
Acid resistance, 15% HCl							
No. of days	105	106	805	8-5	709	623	292
% wt loss	27	26	9	12	7	12	8
Acid resistance, 15% H_2SO_4							
No. of days	49	77	119	77	--	70	77
% wt loss	35	30	26	29		33	26
Abrasion loss							
inches	0.050	0.036	0.015	0.037	0.019	0.026	--
wt loss (g)	14	7	4	6	5	6	--
Cavitation (in.)							
(2 hr exposure)	0.320	0.262	0.020	0.009	--	0.092	0.115

Note: Thermal-catalytically cured. Dried at 221F prior to impregnation.

Table 10.14

Performance of Partially Impregnated Beams
Under Cyclic Flexural Loading[38]

	Control	Surface Impregnated
Load range	280 to 800 lbf	200 to 1,360 lbf
Load rate	5 cycles/sec	3 cycles/sec
Tested cycles without failure	2,000,000	2,000,000
Cracking	None	Small flexural cracks observed at 1,360 lbf load. No spalling.

Table 10.15

Typical Properties of Polymer Latexes for PPCC

Polymer Type	Polyvinyl Acetate	Styrene Butadiene	Acrylic	Polyvinylidene Chloride-PVC Copolymer (Saran)	Neoprene
Percent solids	50%	48%	46%	50%	42%
Stabilizer type	Nonionic	Nonionic	Nonionic	Nonionic	Nonionic
Specific gravity (25C)	1.09	1.01	1.05	1.23	1.10
Weight per gallon (pounds @ 25C)	9.2	8.4	8.8	10.25	9.3
pH	2.5	10.5	9.5	2.0	9.0
Particle size (Å)	N.A.	2,000	N.A.	1,400	N.A.
Surface tension (dynes/cm^2 @ 25C)	N.A.	32	40	33	40
Shelf life	N.A.	>2 years	Excellent	6 months	N.A.
Freeze thaw stability (-15C to 25C)	N.A.	5 cycles	5 cycles	None	N.A.
Viscosity (cps @ 20C)	17	24	~250	~15	10

N.A. = nonavailable.

Table 10.16

Typical Mixture Designs for Latex Modified Mortars and Concretes

Mortar		
	Resurfacing mortar	Underlayment mortar
Sand	250-350 pounds	200-250 pounds
Portland cement	94 pounds	94 pounds
Latex solids[a]	10-20 pounds	10 pounds
Water[b]	50-40 pounds	40-50 pounds
Concrete		
	Mixture 1	Mixture 2
Stone, 3/4-inch maximum	2,240 pounds	1,960 pounds
Sand	1,160 pounds	840 pounds
Portland cement	580 pounds	825 pounds
Latex solids (5-15%)[a]	30-90 pounds	40-125 pounds
Water[b]	250-200 pounds	380-300 pounds

(a) The latex should be formulated with an antifoamer prior to adding it to the mix.
(b) Total water including the water in the latex and the sand.

Table 10.17

Mechanical Properties of Latex Modified Mortars[58,77]

	Control	Styrene-butadiene	Saran[a]	Acrylic	PVAc
Compressive strength (psi)	4500 (5800+)	4800	8430	5700	3700
Tensile strength (psi)	310 (535+) 380	620 830	910	835	700
Flexural strength (psi)	610 (1070+) 820	1430 1730	1820	1835	1840
Modulus of elasticity (10^6 psi)	3.40	1.56	2.25	----	----
Shear bond strength (psi)	50-200	650#	>650	>650	>650

Note: All mixtures had a sand/cement = 3, polymer/cement = 0.20, and were a dry cured 28 days at 50% R.H.
(a) Vinyl chloride-vinylidene chloride co-polymer.
Exceeds shear strength of mortar.
+ Moist cured 28 days.

Table 10.18

Effect of Immersion in Water for 7 Days on the Strength of Latex Modified Mortars[58,77]

	Strength in psi							
	Compressive		Tensile		Flexural		Shear Bond	
	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet
Control	2390	4420	300	310	610	735	40	140
Stryene-butadiene	4950 4800	4100 3680	600 ---	350 ---	1425 1730	925 770	>650#	350
Saran[a]	8430	7150	---	---	1820	1100	>650	>650
Acrylic	5690	5460	835	490	1835	1050	>650	340
PVAc	3750	1300	700	50	1840	320	>650	130

Note: All mixtures had a sand/cement = 3, polymer/cement = 0.2; and were dry cured 28 days at 50% R.H. prior to immersion.
(a) Vinyl chloride-vinylidene chloride co-polymer.
Exceeds shear strength of mortar.

Table 10.19

Durability of Latex Modified Mortars to Chemicals[59]

	Polymer Loading	Percent Weight Loss After 28 Days Immersion in Chemical						
		10% HCl	5% Acetic Acid	5% H_2SO_4	10% NaOH	Xylene	Chlorothene	Methyl Isobutyl Ketone
Control		*	66	40	-5	-4	-3	-5
Styrene-butadiene	15%	54	25	46	-3	*	0	*
	20%	17	10	44	-3	2	0	4
Saran+	20%	44	13	29	-2	-5	-3	-5
	25%	37	8	27	-2	-2	0	-2

* Sample completely destroyed.
\+ Vinyl chloride-vinylidene chloride co-polymer.

Table 10.20

Wear Resistance of Latex Modified Mortars[77]

	Control	Styrene Butadiene	Acrylic	PVAc
Impact strength (in. lb)	6 (7)[a]	19	22	16
Abrasion resistance (% weight loss)	24 (5)[a]	2.5	1.7	5

Note: Mixture had a sand/cement = 3.0, polymer/cement = 0.20, water/cement = 0.50 and was dry cured 28 days at 50% R. H.
(a) Controls were moist cured 28 days.

Table 10.21

Post-mix Polymerized PPCC Compressive Strengths

Reference No.	1	4	45		80		
Curing Regime[a]	R	R	R	T	R	R	T
Polymer loading	8%	10%	4%	4%	5%	10%	5%
Polymer Type							
Control	3525	3820	7830	7830	5285	5285	3980
MMA	--	#	5120	4140	5670	4310	4175
Styrene	--	4270	10,760	10,060	4660	5130	4420
Acrylonitrile	3080	6580	--	--	5315	4255	3735
Vinyl acetate	#	--	--	--	4080	4000	2940
Polyester-styrene	3250	--	--	--	1995	1275	1085

a R = Radiation; T = Thermal-catalytic.
\# Could not be tested due to premature failure.

Table 10.22

Aggregate Gradation for PC[6]

Sieve Size	Percent Retained, %
3/8	29.9
No. 4	19.9
No. 8	5.5
No. 16	5.5
No. 30	9.1
No. 50	8.8
No. 100	5.8
Pan	15.4

Table 10.23

Mechanical Properties of Polymer Concretes

Polymer	Reference No.	Polymer: Aggregate Ratio	Density (g/cm^3)	Strength, psi		Flexural	Modulus of Elasticity 10^6 psi
				Compressive	Tensile		
Polyester	37	1:10	2.52-2.34	15,650-17,800	1700-2000	5000-5700	4-5
Polyester	94	1:9	2.33	9,800	----	2400	4
Polyester-styrene	4	1:4	----	11,650	----	----	--
Epoxy + 40% dibutyl phthalate[a]	37	1:1	1.65	7,100	18,500	----	0.3
Epoxy + polyaminoamide	94	1:9	2.28	9,250	----	3250	4.5
Epoxy-polyamide	37	1:9	1.9-2.1	12,800-14,200	----	4250-5000	--
Epoxy-furan[a]	37	1:1	1.7	8,500-10,000	1000-1150	14	--
MMA-TMPTMA	6	1:15	2.40	19,600	1,430	3100	5

(a) Polymer mortar.

Table 10.24

Properties of MMA-TMPTMA Polymer Concrete at Various Temperatures[6]

Property	Temperature °F	Result
Tensile splitting strength	-15	1,510 psi
	70	1,430 psi
	190	1,370 psi
Compressive strength stress	-15	24,800 psi
	70	19,600 psi
	120	15,800 psi
	190	14,100 psi
Modulus of elasticity	-15	6.11×10^6 psi
	70	5.28×10^6 psi
	190	4.44×10^6 psi
Poisson's ratio	-15	0.24
	70	0.23
	190	0.22
Elastic limit stress	-15	14,000 psi
	70	7,500 psi
	190	4,800 psi
Ultimate compressive strain	-15	5,360 μ in/in
	70	7,080 μ in/in
	190	8,000 μ in/in
Coefficient of expansion	-4 to 70	5.30×10^{-6} in/in/°F
	70 to 140	7.53×10^{-6} in/in/°F

Table 10.25

Chemical Resistance of Polymer Concretes[37]

Concrete Matrix	Relative Resistance on a Scale of 10					
	To Acid	To Oxidizer	To Alkali	To Salts	To Solvents	To Fat/ Petroleum Products
Furan	10	2	9	10	8	8
Polyester	8-9	6-7	3-4	8-10	4-5	7-9
Epoxy	9	3	8	10	6-7	9
Portland cement	1	1	9	5	5-7	5-6

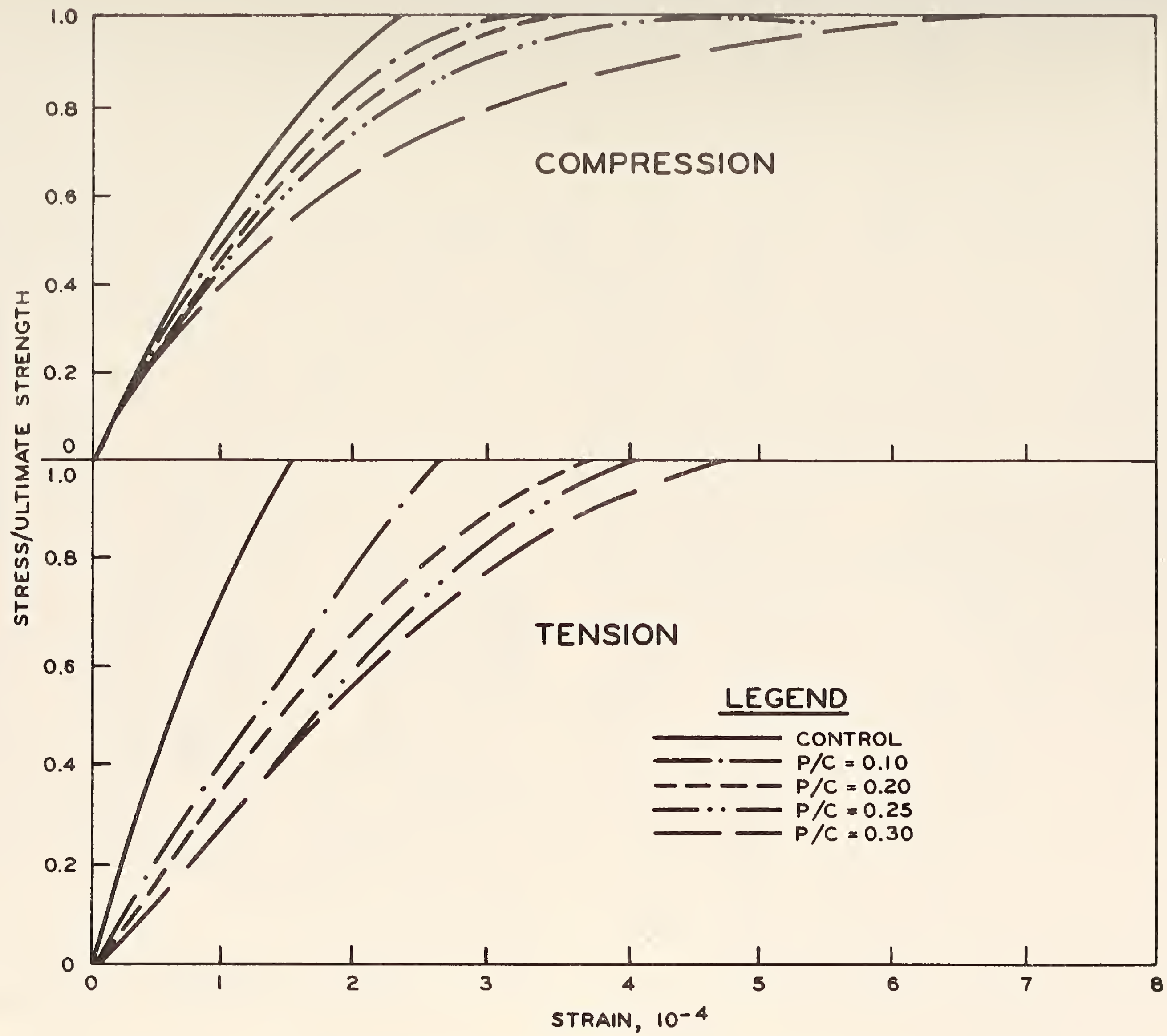

Figure 10.1. Stress-strain curves for polymer modified concrete[78]

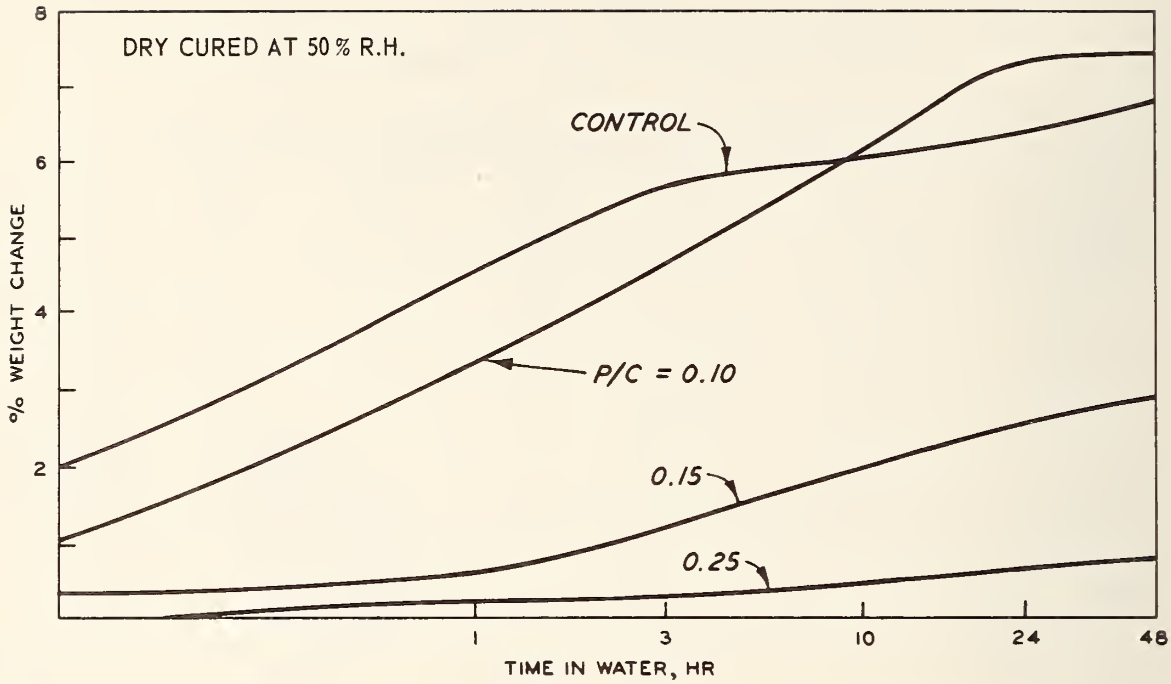

Figure 10.2. Water absorption of polychloropene modified mortar (dry cured at 50 percent R.H.)[37]

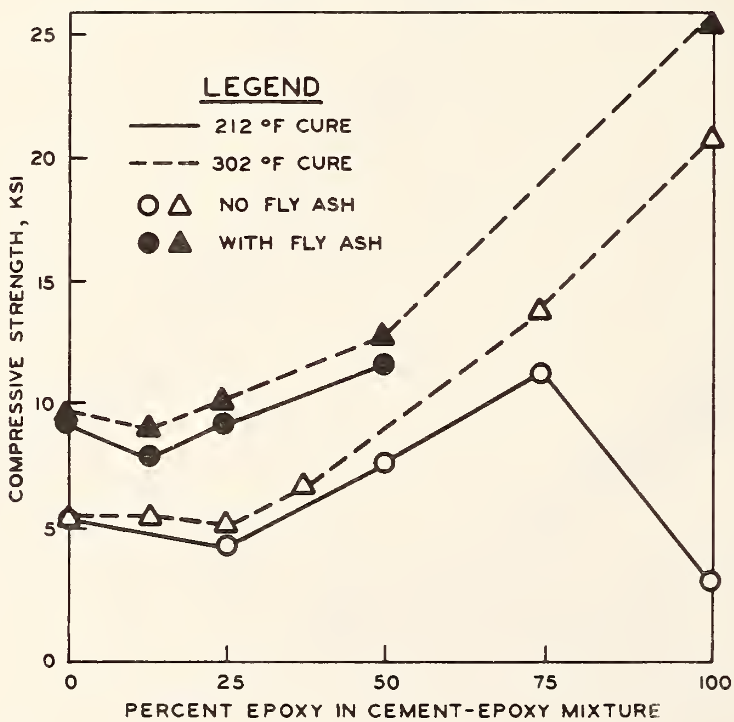

Figure 10.3. Effect of epoxy content on the compressive strength of post-mix epoxy PPCC

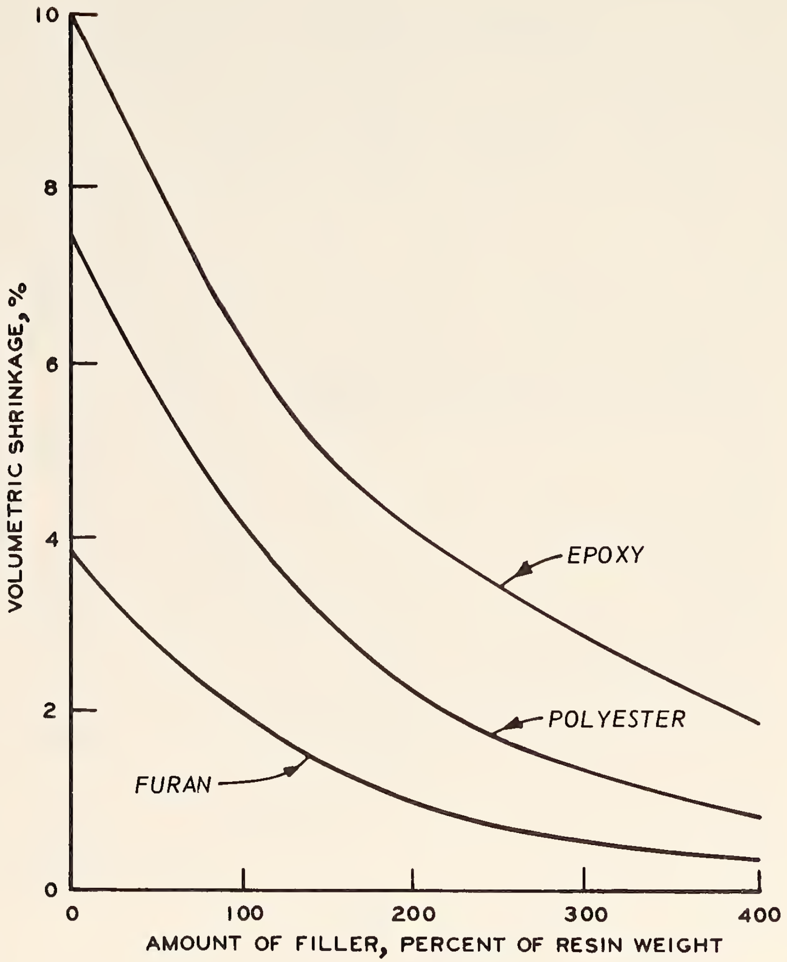

Figure 10.4. Shrinkage versus aggregate content of polymer concrete[87]

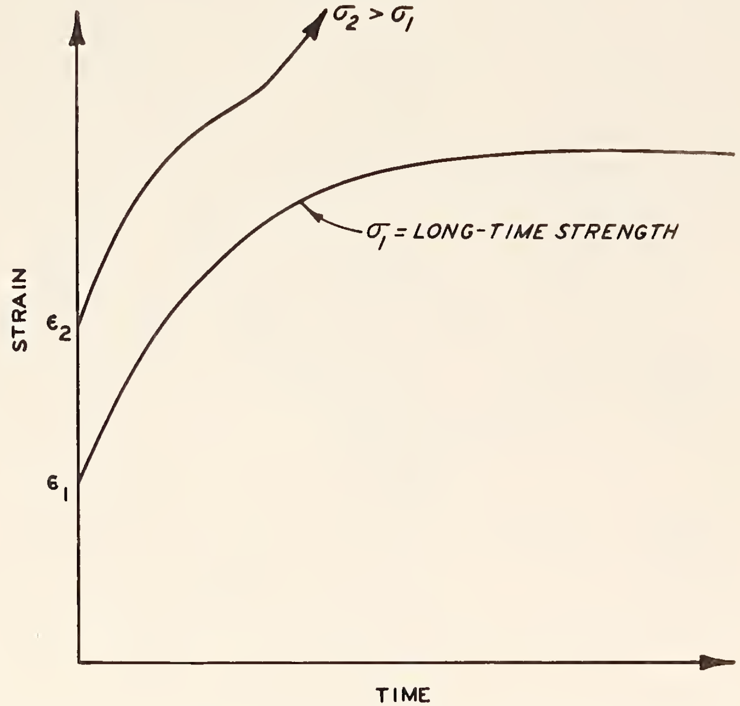

Figure 10.5. Idealized strain-time relationship for polymer concrete at various stress levels[37]

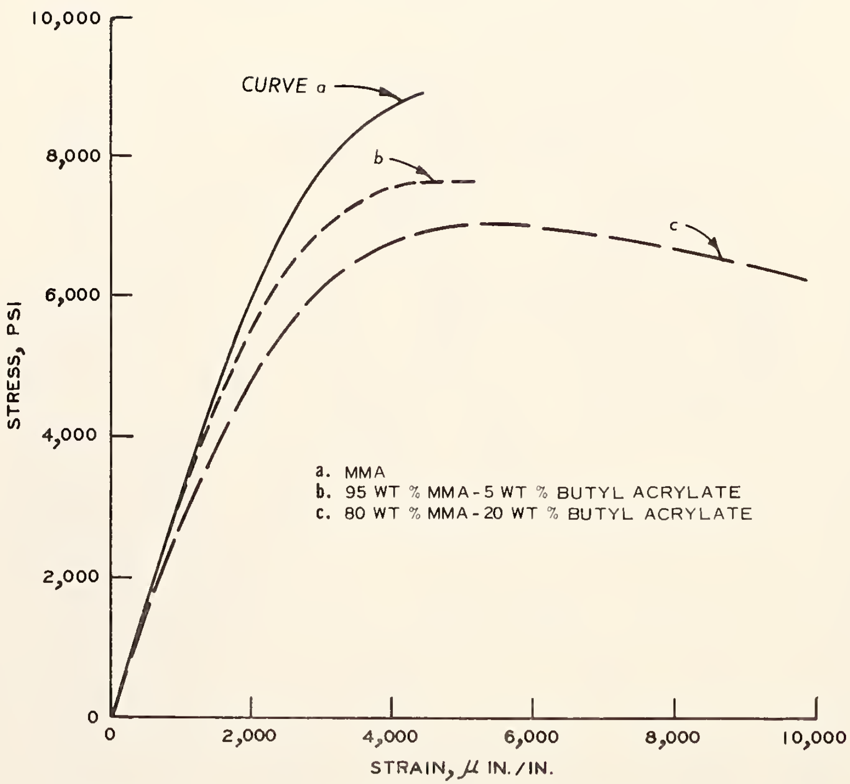

Figure 10.6. Compressive stress-strain curves for polymer concrete[6]

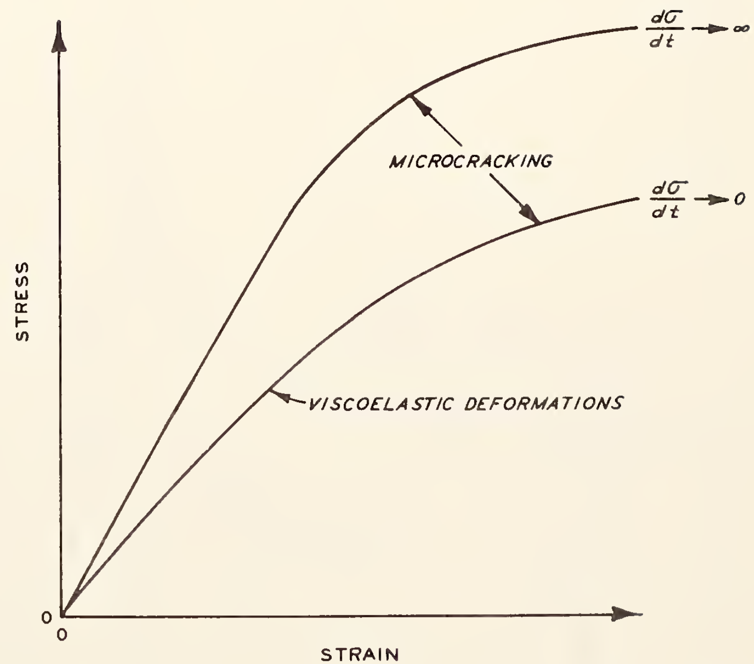

Figure 10.7. Idealized stress-strain relationship for polymer concrete at various stress rates

CHAPTER 11

SEALANT MATERIALS

INTRODUCTION

11.1 BACKGROUND

Concrete pavements on grade and bridge decks have been seriously damaged by traffic and exposure to the environment, and repair costs can be as high as the first cost of the structure.[1] There is evidence that water enters the subbase and subgrade through the pavement, and relatively modest increases in water content can have a dramatic effect on the subbase and subgrade strengths.[2] Damage to concrete pavements caused by penetration of water or corrosive solutions and frost action may be alleviated or even prevented by judicious application of a sealant to the pavement. Sealants can be used to increase the strength of the top surface and reduce wear.

In recent years numerous attempts have been made to solve the dilemma of bridge decks and pavement detrioration through the use of sealants. A sealant may initially be a liquid, sheet, slurry, synthetic or natural material, or a combination of these, and when properly used will form either an impermeable layer on the top surface of the pavement or will seal or fill the internal void system in the concrete. A large number of different types of sealants are available, but only a limited number of investigations have been made in evaluating sealants for pavements (on grade). Numerous state highway departments, foreign countries, and other research organizations have, however, evaluated many types of sealants for protecting bridge decks. This chapter covers those sealants which have been laboratory and field tested by the various organizations and information on sealants furnished by different manufacturers.

11.2 DESCRIPTION OF MATERIALS

Sealants can be grouped into many categories, but for convenience the sealants were grouped into four categories: thermosetting polymers (liquids), asphaltic materials, other resins (liquids), and sheet material. The particular materials reviewed are as follows:

Thermosetting Polymers (liquid)	Asphaltic Materials	Other Resins (Liquid)	Sheet Membranes
Epoxy Resins	Mastic Asphalt	Linseed Oil	Rubber Sheets
Polyesters	Asphaltic Concrete	Silicones	Polymer Impregnated Fabric Sheet
Polyurethanes	Coal Tar	Chlorinated Rubber	Bituminous Coated Membranes
	Rubberized Asphalt	Latex	Modified Coal Tar Reinforced with Fabric
		Waxes	

11.2.1 Thermosetting Polymers.

11.2.1.1 Epoxy Resins. Epoxy resins, since their introduction in 1949, have been widely used due to their excellent adhesion, toughness, and chemical resistance. Epoxy resins are the condensation products of epichlorohydrin with bis-pherol. Epoxy resins systems used as sealants consist of two liquid components (epoxy resin component and curing agent component). There are numerous different amine curing agents available for formulating epoxy resin systems. There are also different flexibilizers, extenders, reactive diluents and fillers which are added to the two components, so that various engineering properties of the cured epoxy resin can be obtained.

11.2.1.2 Polyesters. Polyesters, like epoxy resins, are two component liquid systems and can be formulated to have a wide range of physical properties. Different crosslinking monomers and extenders and catalysts are available for formulations.

11.2.1.3 Polyurethanes. Polyurethanes are among the most recent and promising resins to achieve prominence in the coating field. The elastomeric material is known for high abrasion resistance and durability. Polyurethane coatings are available either as two-component systems that are mixed prior to applications and one-component systems which are moisture cured after application. Polyurethanes may also be formulated to have a wide range of physical properties.

11.2.2 Asphaltic Materials. Asphaltic materials used for pavement sealers include mastic asphalt, coal tar, and rubberized asphalt. These bituminous materials are one-component viscous liquids which are applied hot using trowels, brooms, or squeezes. Coal tar pitch and mastic asphalt for hot application are made by compounding the bituminous base material with filters. Rubberized asphalts are made by adding different types of elastomers (natural rubber, butadiene-styrene and neoprenes) to the bituminous materials.[3]

11.2.3 Other Resins.

11.2.3.1 Linseed Oil. Linseed oil has been the most widely used sealant for pavements. The material is used both boiled and raw and is mixed with water and mineral spirits to form an emulsion. The most widely used form of linseed oil is a 50/50 emulsion with mineral spirits.

11.2.3.2 Chlorinated Rubber. Chlorinated rubber is outstanding in its resistance to water and common corrosive chemicals. It possesses a high degree of impermeability to water vapor. Chlorinated rubber is readily soluble in aromatic hydrocarbons and is manufactured as a spray coating using a solvent.

11.2.3.3 Silicones. Silicones are characterized by outstanding resistance to heat and water and very good weather ability. Silicones are added to the concrete mix before placement and as surface sealers for cured pavements.

11.2.3.4 Waxes. Small particles of montan and paraffin wax are added to the concrete mix before placement. The pavement is heated (185F-212F) after curing to melt the wax. The melted wax fills the capilaries in the concrete and seals the concrete.

11.2.4 Sheet Materials. Numerous impervious sheet materials are available as protective membranes for pavements. The sheet membranes most frequently used are the bituminous types. Below are descriptions of three bituminous types of bituminous sheet membranes:

a. A 65-mil-thick sheet of rubberized asphalt and nontacky bituminous compound reinforced with a woven mesh.

b. A 70-mil-thick sheet composed of coal tar, modified synthetic resins and reinforced with synthetic nonwoven fabric.

c. 80-ml-thick impregnated fiberglass mesh sandwiched between layers of a bituminous mastic and coated with a polyester film. These types of sheets can be spliced together by heat rather than by an adhesive.

MECHANICAL PROPERTIES

The sealants discussed in this chapter were evaluated by many different organizations using many different laboratory test methods. Only a few test results and engineering properties could be found for some of the sealants, however, in other cases large amounts of information was available. The following are some of the properties which are considered by some to be the most important in order for a pavement sealer to perform satisfactorily.[4,5]

a. Adhesion of sealant to substrate.

b. Impermeability of the sealant.

c. Resistance to substrate cracking (flexibility).

d. Resistance to puncture during construction.

e. Durability to moisture and temperature changes.

f. Ease of application.

Other properties such as tensile strength, elongation, compressive strength, hardness, abrasion resistance, tear strength, and water absorption are also important, and some information on these properties was also available for many of the sealants.

11.3 THERMOSETTING POLYMERS

11.3.1 Epoxy Resins. Epoxy resins have been one of the most widely evaluated sealants. Numerous different types of epoxies, coal tar, polysulfate, oil-extended polyamines, and epoxies with nonreactive diluents added, were avaluated as sealants for pavement. Fourteen epoxy systems evaluated by the California Highway Department were laboratory tested.[6] The physical properties of the different epoxy systems are shown in Table 11.1. The range indicates the differences in the epoxy systems tested. Nearly all the epoxy systems evaluated were flexible systems, as indicated by the high elongation and low hardness and tensile strengths.

An electrical resistance test used by both California and Vermont State Highway Departments was used to determine the impermeability of membranes. Resistance tests on impervious material should produce readings in excess of 500,000 ohms/sq ft. Only two out of seven epoxy coatings tested by Vermont State Highway Department exceeded 500,000 ohms/sq ft. Pinholes were observed in most of the epoxy coatings after application. The flexibility of the membranes was determined by bending samples around a mandrel at room temperature and 10F. Most of the epoxies passed the flexibility tests, however, a few became inflexible with time.[7] Epoxy resins were found not to resist cracking as well as other types of membranes.[4]

11.3.2 Polyesters. Very few polyesters have been evaluated as protective coatings for pavements, therefore, only a few formulations were reported. Polyesters like epoxy resins have a wide range of physical properties. The engineering properties of a polyester used as pavement coating (31-830 Polylite), and three polyesters tested by the California State Highway Department are shown in Table 11.2.[6,8] The polyester formulations evaluated produce a flexible coating after curing as indicated by the elongation values obtained. Polyesters do not adhere as well as epoxies to concrete, and a primer coat of epoxy is necessary to obtain the desired adhesion of the coating to concrete.

11.3.3 Polyurethanes. The only physical properties of polyurethanes discussed in various state highway research reports were the impermeability, flexibility, adhesion to pavements, and ease of application. Literature submitted by various manufacturers of polyurethanes did show other physical properties, and these properties are shown in Table 11.3. The polyurethanes listed in Table 11.3 were not evaluated as pavement sealants.

Polyurethanes have been evaluated by a few state highway departments as a bridge deck sealant.[5.9] Both one- and two-component polyurethanes were evaluated. Some of the polyurethanes tested developed bubbles when immersed in water for long periods. Berger[10] reports that polyther-urethanes are much more water and weather resistant than the polyester urethanes. The Vermont State Highway Department measured electrical resistance of pavements coated with polyurethanes and the resistance reading varied from 60,000 ohms/sq ft to 2 million ohms/sq ft, indicating that a primer sheet membrane should be applied before coating the pavement with the polyurethane. The flexibility of the two polyurethanes was found to be satisfactory. The bond bewtween the pavement and the polyurethane coating was found to be poor. Good adhesion of polyurethanes to pavements can only be obtained with the use of a primer.

11.4 ASPHALTIC MATERIALS

Mastic asphalt is more impervious than asphaltic concrete. Blowing and blistering of mastic asphalt, a problem resulting during the application of the material, has been reduced, and in some cases, eliminated by laying it on an open-weave fiberglass fabric.[4,11] Laboratory and field tests have shown that this material is slightly permeable, even when laid on fiberglass fabric.[5]

11.5 OTHER RESINS (LIQUIDS)

11.5.1 Linseed Oil. Some laboratory investigations seem to indicate that the initial treatment with linseed oil emulsions will adequately protect concrete through approximately 50 freeze-thaw cycles. Normally, this sequence is approximately equivalent to one to two years.[12] Surface treatment of nonair-entrained concrete pavements with linseed oil has reduced scaling. Laboratory and field tests have indicated that properly air entrained concrete does not need a linseed oil treatment.[13.14] Field tests have shown that linseed oil emulsions slightly increase the durability of concrete initially, but do decrease concrete durability after seven years exposure.[13] Depth of penetration into concrete of linseed oil is an important property of the material. The penetration depth of linseed solutions according to Steward and Shaffer,[15] is less than one one-hundredth of an inch. Other investigations show that linseed oil emulsions have penetrated concrete 1/16 in. to 1/4 ft in depth.[12] Linseed oil is a temporary sealant and should be reapplied within a two- to four-year maximum time span.[16]

11.5.2 Silicones and Chlorinated Rubber. Silicones and chlorinated rubber sealers have been laboratory tested, and these sealers had no significant effect on the durability of concrete. Silicone and chlorinated rubber coated concrete panels were subjected to freeze-thaw cycles and sealing was not reduced.[13] The coatings were also found not to protect concrete against deicer chemicals.[17]

11.5.3 Modified Latex Concrete. Detailed information on latexes can be found in Chapter 10. Data from the manufacturer of latexes indicate that their products produced a modified concrete with a tensile strength of 570 psi compared to a tensile strength of 410 psi in unmodified concrete. The highest flexural strength obtained for the modified concrete was 1650 psi, and the unmodified concrete showed a maximum flexural strength of 810 psi.

11.5.4 Internally Sealed Concrete. Silicones and waxes have been added to the concrete mixes before placement to seal the concrete internally from moisture and chemicals. Little information was available on the physical properties of the concrete with the silicone additives. Monsanto Research Corporation investigated the addition of a 25/75 montan/paraffin wax to concrete.[18] A conventional concrete bridge slab and a concrete slab with the wax additive were tested for absorbed chlorides. The wax additive prevented the chlorides from penetrating into the concrete slab as shown in Figure 11.1. Compressive strength tests were made on nonair-entrained concrete with the wax additive and conventional air-entrained concrete. The compressive strength (4100 psi) of the internally sealed concrete was slightly higher than the air-entrained concrete (3900 psi). Other properties such as band, skid resistance, abrasion resistance, tensile and flexural strengths, and adhesion are similar to those of conventional concrete. Wax, at 2 and 4 percent by weight of mortar in air-entrained concrete made the concrete completely resistant to scaling through 90 cycles of freezing and thawing. Six bridge decks have since been completed with this material.

A form of internal sealing can also be achieved by forcing or infiltrating a polymer or sulfur into the concrete. Techniques for and characteristics of these approaches can be found in Chapters 6 and 10, respectively, for sulfur and polymers.

11.6 SHEET MEMBRANE

Three different bituminous types of sheet membranes were evaluated by the Vermont Highway Department.[7] These bituminous materials were chosen because of the ease of applications and cost as compared with polymeric sheets of neoprenes, butyl rubbers, etc. The sheet membranes were found to be the most impervious sealants tested by that organization. The flexibility and the ability to resist cracking were good. The bond between the pavement and membrane was acceptable.

USE OF SEALANT MATERIALS

Many different sealants have been used under a wide variety of service and environmental conditions with widely varying results. These results are highly subjective as the success of a sealant under one set of conditions does not necessarily mean it will perform well when the conditions change. The following field use evaluations do give indications of the performance of certain sealants under certain conditions, however.

11.7 THERMOSETTING RESINS

11.7.1 Epoxy Resins. Fourteen different epoxy resin coatings were evaluated in the field by the California State Highway Department.[6] The epoxy resins were applied to two pavement test sections (on grade), one in the valley near Sacramento, and the other at Kinguale located in the mountains. After four years of service the wearing surface at the Sacramento test section was satisfactory except for one section, coated with coal tar epoxy. At Kinguale, a single seal coat application of all epoxies showed considerable wear after one year. The conditions at Kinguale were more severe than at Sacramento. At Kinguale, the epoxy coatings were subjected to extreme temperature changes (0-105F) and to wear from chains and snowplows. Sections of the Bay Area Bridge, Highway 40, and Highway 10 were coated with coal tar epoxy. Both of the sections (on grade) were unsatisfactory after one year, and the bridge deck showed significant wear after five years. The Kentucky State Highway Department[16] used coal tar epoxies to coat four bridge decks. Two of the bridge decks were satisfactory after two years of service and the third deck was unsatisfactory due to bond failures of the epoxy coating. The other bridge deck was found to be unsatisfactory after four years of service.

Seven epoxy resins were applied to bridge decks in Vermont recently.[6] No information is yet available on the long-term durability of these decks. During application many pinholes and blisters were observed in many of the epoxy coatings, and low electrical resistance readings were obtained.

11.7.2 Polyesters. Polyesters behaved very similar to epoxy resin coatings, since both coatings showed more failure in severe climates. A 1/4-in. polyester mortar coating on the Bay Area Bridge in San Francisco showed very little wear after two years of service. Bridge decks coated with the polyester mortar at Alberta and Saskatchewan, Canada, both developed bond failures after two years of service. The polyester coating performed satisfactorily as did the epoxies at the Sacramento test section. Two sections (on grade) at Donner Summit, California, were coated with polyesters. One section was coated using an epoxy primer as binding agents. The section where the epoxy primer was used is showing good durability after three years of service, whereas, the other section does not.

11.7.3 Polyurethanes. Only a few state highway departments evaluated polyurethanes as pavement sealants. The Vermont State Highway Department evaluated two types of polyurethanes.[7] There is no available information on the durability of these materials since they were recently applied. Pinholes and blisters were observed during application indicating that better application procedures are needed. The material did effectively seal the concrete along curb areas. A number of manufacturers that were contacted recommended polyurethane systems and stated that these systems have been satisfactory as sealants for parking decks.

11.8 ASPHALTIC MATERIALS

The Kentucky Department of Highways surfaced four bridge decks with bituminous materials.[16] The general performance of the bituminous surfaces was found to be rather unsatisfactory. Inspections of the decks indicated that the overlay leaks, hinders drainage, traps salts, and delays drying. It was also found that it is essential that a bridge deck be adequately sealed before applying a bituminous surface.

Vermont Department of Highways evaluated a hot applied rubberized asphalt and coal tar applied to woven glass fabric as bridge deck sealants.[5.9] Both overlays were found to be disappointing and were not recommended for further use as a bridge deck sealant. The hot mopped rubberized asphalt, when applied over glass fabric, formed bubbles and pinholes and did not sufficiently seal curb areas. The flexibility of the material decreased by overheating. The flexibility of the coal tar was not sufficient to resist cracking, and delaminations were noted when the coating was exposed to water and freeze-thaw cycles.

Only a few field evalutions of mastic asphalt as a pavement sealant have been made. Blowing and blistering of the mastic asphalt, a problem applying the material, has been improved by laying it on open-weave fiberglass fabric.[4,11]

11.9 OTHER RESINS (LIQUIDS)

11.9.1 Linseed Oil. Although linseed oil and mineral spirits is the oldest and most widely used protective coating for concrete, it has been shown to have varying effects on concrete from climate to climate. In warm climates where the surface was prepared properly, two coats of a 50/50 mixture normally seals the pavement adequately.[14] Proper application of the material acts as a seal, but after two years' exposure to moisture, chlorinated salts, and traffic, an additional application of a second coat is need to seal the concrete.

Since it would be impossible to give every example of the use of linseed oil as a seal coat, the following examples may be noted:

a. The Kentucky Department of Highways reported[16] that boiled linseed oil should be applied at two- to four-year intervals. Periodic inspection should be made in order to detect deterioration. Effectiveness of linseed oil treatments to older structures is rather difficult to evaluate, however, numerous inspections indicate that the coating has been successful in arresting deterioration. All decks in Kentucky coated with the linseed oil protective treatment showed signs of premature sealing.

b. The Texas Transportation Institute reported[14] linseed oil and epoxy treatments were the most effective in preventing water absorption. Mixtures of linseed oil, kerosene, and tung oil provided considerable protection against freeze-thaw sealing if applied without delay after curing. Little protection benefits resulted from the application after the concrete had begun to scale.

c. The Department of Highways, Ontario, Canada, evaluated six different materials on two test sections (one made from air-entrained concrete and one made from nonair-entrained concrete) near Bronte, Ontario.[13] A 50/50 linseed oil emulsion was one of the materials evaluated. Inspections of the test section were made after two, five, and seven years of use. When compared to untreated control concrete, the general effect of the linseed oil treatment was to marginally increase the durability up to two years and to significantly decrease the durability at seven years for air-entrained concrete. Significant deterioration occurred on the section of nonair-entrained concrete. None of the surface sealers applied to this concrete had a significantly beneficial effect.

11.9.2 Silicones and Chlorinated Rubbers. Silicone and chlorinated rubber surface sealing compounds were found to have little effect on the resistance of scaling of nonair-entrained concrete. Neither surface sealer had a significant effect on the durability of concrete pavements (on grade). The pavements treated with the materials performed in a similar way to untreated pavements.[13] A silicone admixture which was evaluated by the Kentucky Department of Highways on a bridge deck delayed initial set by approximately 40 hours and promoted considerable bleeding.[16] The deck was found to be in excellent condition after two years, however.

11.9.3 Latex Modified Concrete. Since 1961, latex modified concrete has been used in the construction of bridge decks. Maine, Pennsylvania, and Kentucky Highway Departments have evaluated the material.[11] The latex modified concrete was claimed to improve toughness and improve bond between the mortar and old concrete. To date performance of the surfacing has been fairly satisfactory, although numerous shrinkage cracks are evident. Some Kentucky structures have been in service for five years with satisfactory performance.

Ten decks have been constructed in Pennsylvania since 1966 using a 1-in. latex modified concrete overlay. These decks have only been open to limited traffic and therefore do not show full service use to determine suitability of the material. The overlays have performed satisfactorily with the exception of two decks. The failure of these decks may be due to improper surface preparation.[19]

Latex modified concrete overlays for bridge decks have proven successful in Maine.[20] One bridge deck was overlaid with the latex concrete in 1959 and the other in 1969. A 1972 field inspection found both decks in good condition.

11.10 SHEET MEMBRANES

Sheet membranes have been evaluated by several different organizations. These bituminous sheet membranes have been found to be impermeable to water and salt solutions.[5,7] In recent years several types of bituminous sheet membranes have been applied as protective coatings for bridge decks. These sheet membranes were coated with a bituminous concrete top coat. No conclusive information on the durability of these membranes was reported. Some have shown considerable promise while others have failed due to cracking and poor adhesion to the top coat.

In addition, these sheet membranes have performed poorly as edge seals to curb areas.

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CHAPTER 11

SEALANT MATERIALS

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Table 11.1

Physical Properties of Epoxy Resins[6]

Property	Test Results: Average*-1	Test Results: Range
Viscosity (poises)*-2	50	28 - 72
Tensile strength (psi)	916	453 - 3655
Elongation (%)	52	10 - 104
Compressive strength (psi)	3057	1277 - 10,231
Hardness (Shore D)	45	28 - 77
% Water absorption	0.64	0.17 - 2.1
Tensile adhesion (California 420A) (psi)*-3	273	143 - 318

*-1 The average of all fourteen epoxies tested.
*-2 Only two epoxies tested.
*-3 Concrete failures were observed for most specimens.

Table 11.2

Physical Properties of Polyesters[6,8]

Property	Formulations Evaluated: 1	2	3	4
Viscosity (cps)	750	800	1000	--
Specific gravity (g/cm^3)	1.14	--	--	--
Tensile strength (psi)	2800	3960	1720	900
Elongation (%)	60	23	63	85
Compressive strength (psi 28 days)	4780	5100	3240	--
% Water absorption	--	0.75	1.3	1.2
Tensile adhesion (California 420A) (psi)		174*	143*	

* Bond failed.

Table 11.3

Physical Properties of Polyurethanes

Property	Polyurethanes RRC-1654	Elastuff 504	M413	Poly-Q
Hardness, Shore A	80	90	--	95
Tensile strength (psi)	2500	2400	5000	2200
Elongation (%)	300	310	800	350
Tear strength (pli)	--	300	--	300
Abrasion resistance (Taher CS-17 wheels 100 cycles)	6.0	12.0	--	--
Temperature limits (high F)	--	200	--	250
Temperature limits (low F)	--	-70	--	-50
Water vapor transmission (ASTM E96-66, perms/20 mils)	--	0.86	--	--

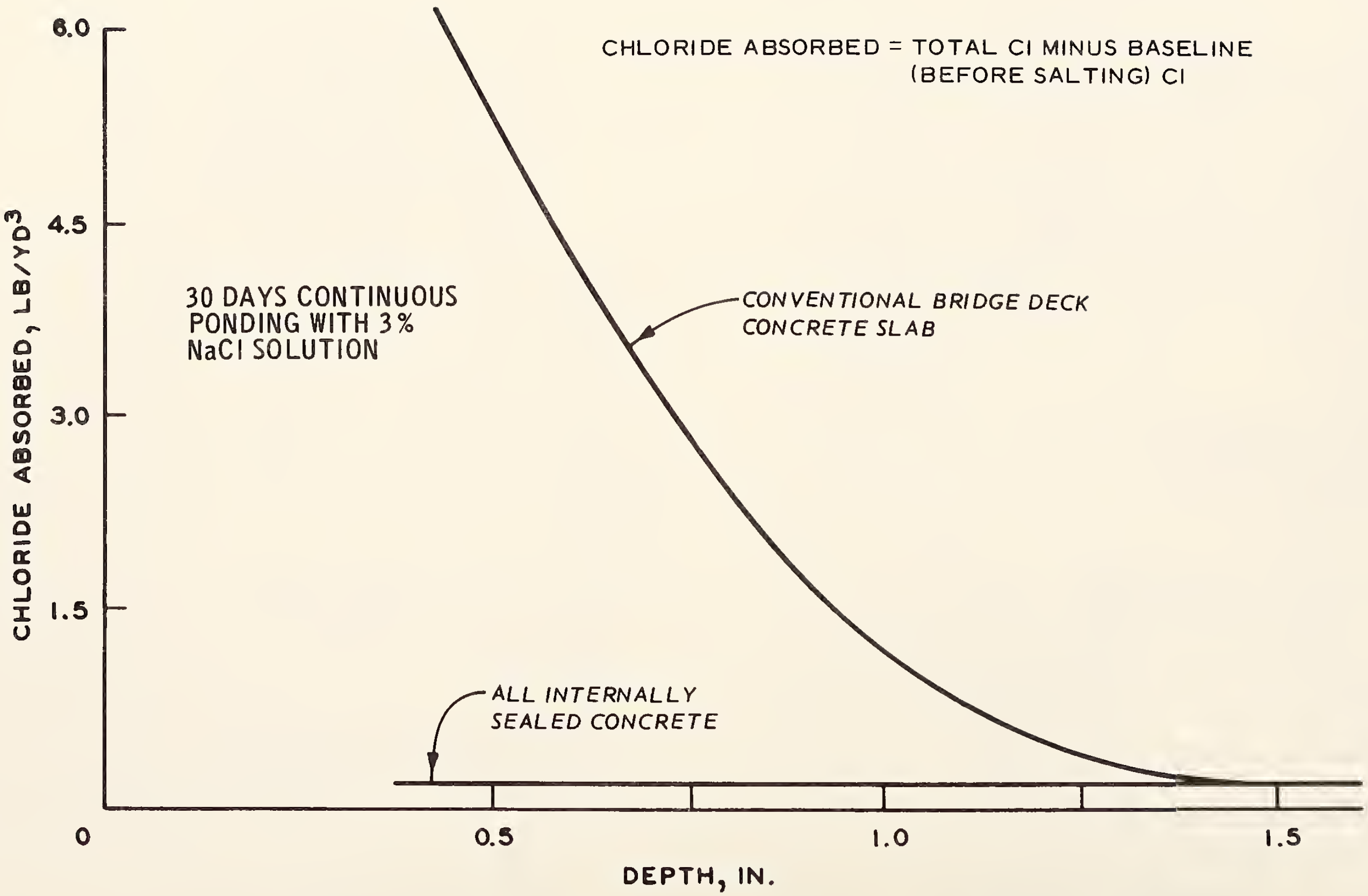


Figure 11.1. Effect of internally sealed concrete on depth of penetration of chlorides[18]

CHAPTER 12

CERAMIC MATERIALS

INTRODUCTION

12.1 BACKGROUND

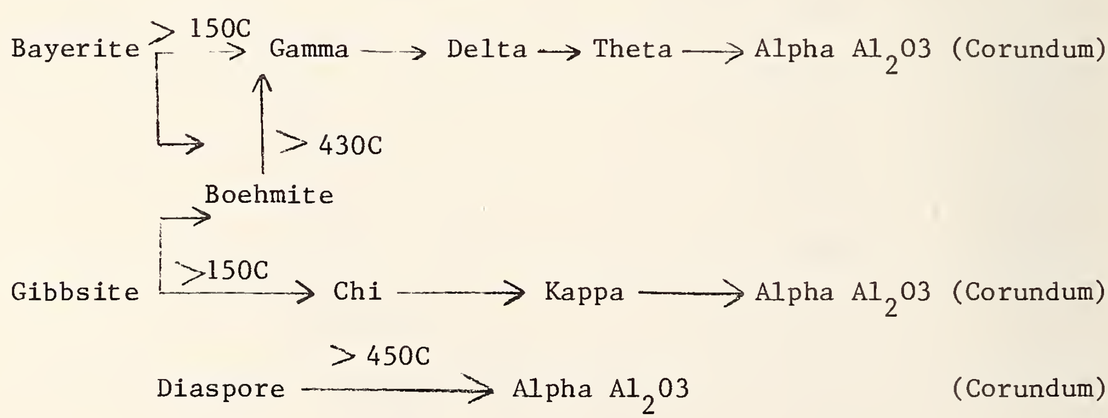
Ceramics is classically defined as the technology and "science of making and using solid articles which have as their essential component, and are composed in large part of, inorganic nonmetallic materials. This definition includes not only materials such as pottery, porcelain, enamels, cements, and glass, but also nonmetallic magnetic materials, ferroelectrics, manufactured single crystals, and a variety of other products which were not in existence a few years ago." The potential for developing new "ceramics" is also very great. Modern developments in methods of fabrication, the use of materials to close specifications, and their new and unique properties often make traditional definitions too restrictive and for the purposes of this chapter, ceramics must be defined by using a fundamental approach to the art and science and a broad view of what ceramics is.[1]

Ceramic processes have had an impact on pavement technology in the United States ever since pozzolans were used in experimental highways as early as 1938 in Illinois, Nebraska, Michigan, and Wisconsin[2] and in 1949 in the McPherson, Kansas, test road.[3] Fly ash is regularly specified in Alabama and is an optional alternate for highways in Georgia.[4] Pavements for the US Air Force have been built in the German Federal Republic with portland blast-furnace slag cement (hochojen zement). Calcined bauxite has been used in polish-resistant road surfacings in England since 1959.[5,6,7] Other ceramic products are possibly useful in zero-maintenance pavements and are considered and evaluated in this report.

12.2 DESCRIPTION OF MATERIALS

<u>12.2.1 Calcined Bauxite</u>. Calcined bauxite has promise in both a neglected and a well investigated area. The neglected area is in its capacity for producing highly durable textured surfaces, and the investigated area being the capacity for producing highly skid resistant surfaces. Bauxite is an aluminous laterite, which may or may not be hard seed coherent. It is the major ore of aluminum and occurs in many countries, in many kinds of residual deposits, and can come from many geologic ages.[7] The aluminum minerals in bauxites are bayerite, gibbsite, and hardstandite, which are naturally occurring polymorphs of $Al(OH)_3$ and diaspore and boehmite, which are naturally occurring polymorphs of AlO(OH).[8] To some degree corundum (stable Al_2O3) occurs in some bauxites.

Bauxites contain other minerals such as hydrous iron oxides, iron oxides, aluminous clay minerals, and other clay minerals; consequently, the products of calcining bauxite include glassy builders softer than the major hard constituent, corundum, which has a Mohs scale hardness of 9 and a Knoop hardness of 2000 (Diamond has a Mohs scale hardness of 10 and a Knoop hardness of 7000).[9] The sequence of alumina formation from bayerite, boehmite, gibbsite, and diaspore has been shown as[10]



When calcined bauxites have been examined by x-ray diffraction after calcining at 1500C, it is noted that cristobalite or quartz and kaolinite are likely to react to form mullite[11] during that calcination process. Based on trials in the Transport and Road Research Laboratory's modified rotating steel-making furnace, the best calcination methods found so far are to charge the cold furnace with 50 to 100 kg of air-dry raw bauxite of known grading and then raise the temperature to 950C in about 30 minutes. The heating rate is then increased to a higher level depending on the bauxite used and held there for 30 minutes before the furnace flame is extinguished. The higher temperatures have been found to be 1450C, 1520C, and 1575C for Northern Irish, Ghanian, and Australian bauxites, respectively. The charge is then initially cooled in the rotating furnace and finally cooled in open air. Calcination temperatures are critical for the several bauxites. The optimum appears to be the temperature where the surfaces of the particles fuze slighly but do not adhere to the furnace linings. Too low a temperature means a friable product while too high a temperature makes a smooth glassy material which adheres to the furnace lining. The literature found did not indicate that tests of Jamaican bauxite had been made, although Jamaica is a major source of bauxite. Experimental calcination procedures can improve the quality of the calcined bauxite produced using other raw bauxites.[6,11]

Tests in the United Kingdon have established that a RASC grade calcined bauxite from Guyana is the most desirable grade in terms of high resistance to polishing and maintenance of desirable surface textures. Because this material is also available in large sizes, it ia also used as aggregate thus causing the quantity then available from Guyana to barely meet the refractory requirements in England.

12.2.2 Pozzolans. In the United States, fly ash, the ash of pulverized coal burned in electric generating stations, is the most widely available and used pozzolan. Because of regulations controlling the permitted emissions of SO_2 and other polluting gases and air-borne solid wastes, characteristics of fly ashes are changing as lignite and sub-bituminous coals are burned in place of the bituminous coals burned in the past. These two coals result in less emission of SO_2 than the bituminous coal. Some lignite fly ashes have calcium oxide contents such that mortar cubes with no addition of $CA(OH)_2$ have compressive strengths above

3000 psi at 28 days; they are apparently not sulfate-resistant, however, as indicated in tests underway at the Bureau of Reclamation.[12] Fly ashes meeting the current specifications of the US Army Corps of Engineers may be expected to contribute to sulfate resistance of concrete. Fly ashes of this type are candidate materials for zero maintenance pavements in circumstances where economy is important, or during cement shortages, or in circumstances where the economically available aggregates are known to be capable of deleterious alkali-silica reaction. Pozzolans may also be useful in cases where highways pass over soils containing concentrations of sulfate or where the groundwater contains sulfates in amounts making sulfate attack upon the concrete highway probable. In parts of the western United States, the first requirement for producing a zero maintenance pavement is likely to be the use of sulfate-resistant portland cement concrete or of a bituminous concrete. Sulfate resistance can be increased by the use of pozzolans in portland cement concrete as an alternate to the use of Type II or Type V cement. Some experimentation in this area is warranted.

12.2.3 Portland-Blast Furnace Slag Cement or Granulated Slag As A Mineral Admixture. Portland blast-furnace slag cement is essentially an intimately interground mixture of portland cement clinker and granulated blast-furnace slag or an intimate and uniform blend of portland cement and fine granulated blast-furnace slag in which the amount of the slag constituents falls within specified limits. Granulated blast-furnace slag is a nonmetallic product consisting essentially of silicates and aluminosilicates of calcium which is developed simultaneously with iron in a blast furnace and is granulated by quenching the molten material in water or in water, steam, and air. These materials can be useful for the same situations described in Section 12.2.2.

12.2.4 Expanded Clay, Shale, and Slate As Lightweight Aggregates. Lightweight structural concrete has been used with varying success in bridge decks of highway bridges. The improved dead load to total load relation and the improved thermal characteristics of structural lightweight aggregate concrete are the primary characteristics in selecting lightweight aggregate concrete for a project.[13] While lightweight aggregate concrete may be useful in structures ancillary to a highway, it appears as an unlikely candidate for use in the pavement itself. The rough surface that might be achieved by using a no-fines lightweight aggregate concrete as a wearing surface would, like other no-fines concrete in pavements, silt up and lose drainage and roughness that otherwise would increase wear resistance. These disadvantages have been demonstrated in no-fines concrete made with stronger aggregates.[14]

12.2.5 Cement Clinker as Coarse Aggregate. Berger[15] reported tests in which Type I portland cement clinker as coarse and fine aggregate was compared with glacial gravel and sand in concrete test specimens. Although the clinker aggregates produced high strengths in compression, tensile splitting, and in flexure, concrete containing clinker coarse and fine aggregate showed more expansion on soaking than concrete containing natural gravel and sand. Concrete specimens dried at room temperature after curing developed surface popouts. Consideration of cement clinker aggregate is not recommended as a candidate material for zero maintenance pavement, because cement clinker is inherently a material providing volumetrically unstable aggregate.

12.2.6 Refractory Shapes. Tests have been made of several refractory shapes in test tracks in which they have been compared with conventional concrete. The properties of the shapes include hardness like that of calcined bauxite with higher modulus of rupture and higher modulus of elasticity. The shapes provide an extremely durable wearing surface and may be of possible use and economically justifiable for very severe wear situations. Dr. Della M. Roy provided information on these tests and of the availability of some of the refractories as crushed coarse aggregate.[16] Properties of these refractories were not available.

12.3 CANDIDATE MATERIALS

The most promising ceramics for candidate zero maintenance materials appear to be:

a. Calcined bauxite to be used in top courses to resist wear.

b. Pozzolans, especially fly ash and calcined shale, under circumstances where resistance to alkali-silica reaction or sulfate attack are needed and low alkali cement and sulfate-resistant cements are not available.

As the latter is a more specialized case, only calcined bauxite, which has a broader area of use and which has not been as widely researched as the pozzolans, will be reviewed in the remainder of the chapter.

CALCINED BAUXITE

12.4 PHYSICAL CHARACTERISTICS

The static modulus of elasticity of polycrystalline alumina ranges from 30 to 56 x 10^6 psi. That of calcined bauxite can be expected to be lower. The compressive strength of polycrystalline alumina ranges from 260,000 to over 400,000 psi where the porosity is 2 to 20 percent. Bending strength of polycrystalline alumina ranges from 20,000 to 36,000 psi at room temperature, diminishing with increasing porosity. The tensile strength of polycrystalline alumina, 95 percent dense, is about 22,000 psi at room temperature. Load-deformation characteristics were not located, nor was fatigue strength information. Permeability of low-porosity calcined bauxite is expected to be very low. The properties reported above are largely those of pure polycrystalline alumina, which will be higher than the unreported properties of calcined bauxite.[17]

The abrasion resistance is high; the Mohs scale hardness is 9 and the Knoop indent hardness is 2000. Alumina with minor chromium or iron substitution is ruby and alumina with minor iron or titanium is sapphire. Both are hard gemstone. Resistance to chemicals is described as "very slightly soluble" in acids and alkalies. Alumina is a refractory melting above 3500F and is therefore fire resistant. It is, in fact, incombustable. The thermal coefficient of pure 1_2O_3 in temperatures from ambient to 1000F is about 0.3 percent±0.1 percent.

When calcined bauxite sand is used in concrete, dimensional changes in the unhardened state should not differ greatly from those of concretes with ordinary natural aggregates of high elastic modulus. Little shrinking or swelling is to be expected in the mortar or concrete and little creep is to be expected, as is true in concretes made with other aggregates of high elastic modulus. Durability to freezing and thawing in concrete is expected to be good.

Two British tests that appear to be useful in predicting the behavior of calcined bauxites of a given maximum size in pavements are the polished stone value and the aggregate abrasion value.[18] High numbers of the polished stone value indicate that the aggregate maintains its surface texture and resists polishing. Low numbers of the aggregate abrasion value indicate that the aggregate is tough and highly resistant to abrasion. The polished stone value for RASC Guyana calcined bauxite was 75 while the aggregate abrasion value was 3.0.[11]

Examination by scanning electron microscope showed that the Northern Ireland, Ghanian, and Australian calcined bauxites crushed after calcination had rough textures in which small rounded crystals of corundum protruded from a glassy matrix. The examination did not show the well-shaped crystals characterizing good calcined bauxites found in earlier tests of Guyanan RASC grade.[11] Nevertheless, 5mm nominal size laboratory calcined Ghanian bauxite had road-trial ratings as good as 3mm commercial RASC grade calcined bauxite and something more than twice as deep a texture. The 3mm calcined Ghanian bauxite and the two sizes of calcined Australian bauxites were also very promising in the road trials.

12.5 EASE OF CONSTRUCTION

Calcined bauxite is usually graded as a one-size sand (5mm or 3mm) and can be used in a normal concrete or mortar mixture proportioned to tolerate the calcined bauxite admixture. It should be possible to make concrete or mortar of high strength in compression or flexure and to use admixtures normally used in good highway practice. Mixing and curing should be as for normal concrete or mortar mixtures. Calcined bauxite concretes or mortars should tolerate any kind of curing used in good highway practice or in precast work. Calcined bauxite in mortar wearing courses or as chippings in asphaltic concrete with a chipping surface should be easily used in construction but may cause unusual wear of equipment.

12.6 USE OF CALCINED BAUXITE

12.6.1 Resin-Based Toppings. Initial road trials have been made[6] by spreading shell-grip binder (an epoxy formulation) on outlined panels. It was distributed by hand but drawn from a bulb distributor to be certain that it was at correct temperature, homogeneous, and correctly proportioned. The aggregates were hand-spread using a board as a baffle and straight edge to prevent contamination between adjoining panels. The site of the test was on a sharp bend where there were 16,000 turning vehicles a day. It had been demonstrated that one winter followed by one summer at this site yields meaningful conclusions. Most of the polishing or loss of texture takes place in the summer. Results of several other tests of a variety of materials reiterated the superior durability and retention of texture and road surface obtained by the use of calcined bauxite in resin as a topping for a concrete highway.[6] For usage in the United States it is not believed that it would be desirable to seal the top of a pavement such as a layer of resin would do because of the undesirable effects in increasing water saturation of the pavement, base, and subbase. The evidence of durability of these surfaces and their retention of high skid resistance is relevant to the zero maintenance concept and should be given further consideration.

12.6.2 Asphaltic Concretes. Experiments have been conducted with calcined bauxite in surface dressings bound with tar and bitumen[19] with and without coarse aggregate and in rolled asphalt containing 12.5mm chippings and roadstone. In the surface dressing the calcined bauxite retained the highest summer skid resistance value although the texture depth was fairly low. In tests with 30 percent of a coarse aggregate, the sidewise friction coefficient and summer skid-resistance values were again the highest in this series of tests as in the previous series of tests. The conclusions included the following concerning the performance of the bauxite:

a. Providing adequate road surface texture is maintained, reducing the nominal maximum size of an aggregate raises the resistance to skidding of a surfacing made with it. Over the range of sizes studied (3mm to 25mm nominal sizes), halving the size of aggregates used as chippings increases the sidewise friction coefficient by about 0.08 units: In the case of macadams, the corresponding increase is about 0.03 units.

b. There is some indication that the use of more nearly single-sized chippings in surface dressings will give a better surface texture.

12.6.3 Portland Cement Mortars and Concretes. Stingley, et al.[3] described a series of evaluations of concrete specimens containing calcined bauxite where the surfaces of the concrete specimens were textured 30 minutes after casting using either a wire broom or a rubber-tined rake. The specimens were cured under cover for 24 hr and then in water at 68±4F until 28 days. Cubes of concrete and mortar were made from the same mixtures and tested in compression at 28 days. Mortars with calcined bauxite sand contained either 1.5:1 or 1.3:1 sand:cement and had high compressive strengths. Tests were made of concrete containing the following aggregates:

a. Natural coarse aggregates and a chert sand (two strength levels of concrete: 5800 and 7280 psi).

b. Natural coarse aggregate and crushed gritstone fine aggregate.

c. Ten commercially available limestones and their local sands.

d. Ten commercially available limestones and a flint gravel sand from the Thames valley.

e. Nine coarse aggregates representing aggregates with polished stone values from 32 to 72.

f. Calcined bauxite in a grading from 1/2 to 3/8 in.

In mortars, sand manufactured from flint gravel, dolomitic limestones, dolerite, and gritstone along with calcined bauxite fines and natural flint gravel sand were evaluated. All specimens were tested with an accelerated wear machine in which each specimen was exposed for 50 hr to abrasion by two rubber tired wheels turning against the concrete specimens. The testing machine is described and illustrated in Reference 21.

The angle of scuff between the specimens and the tired wheels was 2. The tire pressure was adjusted to give running pressures of 34.8 psi and contact pressure of 27.6 psi. The machine feeds dry flint gravel sand graded between 2.4mm and 1.2mm at 2.0 kg per hour for 50 hr of wearing period, followed by 5 hr of wet polishing with 0.25 kg/hr of fine emery flour lubricated with water fed at 1.4 cubic decimeters per minute.

As noted above, textures were applied to the surfaces of the concrete specimens by brushing with a steel broom and by tining with a rubber-tined rake. Tined surfaces had deeper textures and less regular projections than brushed surfaces and maintained higher skid resistance value after the accelerated wear test.[21,22] Several conclusions of Reference 22 are important to the zero maintenance concept:

> "2. The hardness of the sand, i.e., its resistance to abrasion was of major importance. The harder sands generally yielded better skid resistance values at the end of the dry wearing stage of the test because they were predominant in the surface. During the period of wet polishing, the skid resistance value of mortars made with the harder sands decreased as the sand particles became polished and those of mortars made with softer sands increased as the cement paste lost its polish.
>
> The very hard and polish-resistant calcined bauxite was always superior to the other sands.
>
> . . .
>
> An increase in the coarse fraction of the sand reduced the loss of texture during the test.
>
> . . .
>
> 3. Significant relationships between the polished-stone value of the coarse aggregate and skid resistance value were obtained but the effect was small; an increase in polished-stone value from 35 to 72 yielded an increase in skid resistance value after the full cycle of test of less than 5 units. The exposure of the coarse aggregate, however, was always low and never greater than 12 percent of the total surface area . . .

4. There was an observed general tendency for mixes of higher strength to yield lower skid resistance values after wear, although the trends were not always significant.

5. The hardened cement paste became highly polished during dry wear but this polish was removed in the presence of water. This effect was most marked in mixes incorporating fine aggregates having less resistance to abrasion. In these cases, more hardened cement paste made contact with the pendulum slider.

6. The accelerated wear test used in these tests was more severe than the polished-stone test. Further consideration, however, must be given to the test before it can be satisfactorily used to specify acceptance limits for particular materials."

It also seems to be reasonable to expect that preservation of the top surface under traffic wear, or lengthening its life, will tend toward realizing a zero-maintenance pavement system.

12.7 EASE OF MAINTENANCE

The maintenance, if ever needed, of pavements containing calcined bauxite should be no more difficult than maintenance of a standard asphalt or concrete pavement.

12.8 ENVIRONMENTAL COMPATIBILITY

There is nothing to indicate that the addition of calcined bauxite to a pavement would change the degree of that pavement's compatibility with the environment.

12.9 AVAILABILITY AND COST

Bauxite is available from many sources, most of them outside the United States. Calcined bauxite presently is important as aggregate for castable refractories. The cost of calcined bauxite was quoted as 45 Pounds Sterling per ton in 1974. Assuming an exchange rate of $2.50 US per Pound Sterling and an inflation rate of 20 percent, the cost of calcined bauxite in the United States should be in the vicinity of $135 per ton.

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CHAPTER 12

CERAMIC MATERIALS

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CHAPTER 13

PRESTRESSED CONCRETE PAVEMENTS

INTRODUCTION

13.1 BACKGROUND

The imbalance between tensile and compressive strengths in a concrete element can be decreased by prestressing. The precompression in the concrete is cumulative with the inherent flexural strength of the material to produce an increase in stress range in the flexure zone. Prestressed concrete pavement introduces internal stresses of such magnitude and distribution in the concrete that tensile stresses resulting from service loads are counteracted to a desired degree.

Challenging secondary advantages of prestressing in pavements are:[1] (1) elimination of a large percentage of transverse joints except at structures, (2) absence of cracks in the road surfaces, (3) resultant reduction of moisture in the road foundation, (4) warping and curling stresses are decreased in proportion to decrease in thickness, and (5) slab thickness is reduced.

13.2 EUROPEAN EXPERIENCE

Research and experimental work on prestressed pavement has been underway for about 30 years, primarily in Europe. Variables have included:[1,2,3,4]

a. Slab lengths: ranging from 100 ft to over 1000 ft.

b. Thickness: from 2-1/4 in. to 10 in.

c. Longitudinal prestress: from 90 to 1850 psi.

d. Transverse prestress: from 0 to 125 psi.

e. Prestressing techniques: three basic methods.

f. Types of joints and connections.

g. Frictional forces between slab and subgrade.

The state of the art of prestressed concrete pavements was presented quite comprehensively in the report of the Subcommittee on Prestressed Concrete Pavements of the HRB Committee on Rigid Pavement Design.[5] Practically all prestressed concrete pavements up to the present time are to be found in Western Europe and most of these were constructed in the late 1950's.[6] Except for reports that these pavements are performing satisfactorily, little detailed performance data concerning them are available.

Design theories and construction practices have been developed to the point where prestressed pavements can compete economically against conventional concrete pavements in certain countries in Europe--especially on airports where the reduction in pavement thickness can be more than 50 percent. This reduction could probably apply to highway pavements as well, but because of necessary construction procedures, a minimum thickness of about 5 in. seems to be required.[7]

13.3 UNITED STATES EXPERIENCE

To date (1975) there have been four significant installations of prestressed concrete pavement in the United States. In 1956, the Jones and Laughlin Steel Company built a 5-in.-thick, 12-ft-wide, 400-ft-length of pavement on their installation at Pittsburgh, Pennsylvania.[8] In the spring of 1971, a 14-ft-wide, 6-in.-thick, 300-ft-long slab was constructed by the Delaware Department of Highways and Transportation at Milford, Delaware.[9] These projects, though successful, were too short in length and the alignment characteristics were not of such a nature as to provide a firm basis for the development of valid design criteria and construction methods needed in constructing prestressed concrete pavement on a larger, more practical scale.

A prestressed concrete highway pavement was constructed in late 1971 at the Dulles International Airport near Washington, DC. The pavement is 3,200 ft long and consists of six slabs ranging in length from 400 to 760 ft. The concrete is 6 in. thick and 24 ft wide.[10] Measurements made during construction and for 3-1/2 months thereafter included concrete strengths, strand elongations during stressing, concrete temperatures and gradients, slab movements, and profile changes. Results indicate that concrete pavements of normal alignment and grades can be easily prestressed to 200 psi at ends of long slabs by post-tensioning steel, using commercially available prestressing strand products, hardware, and jacking equipment.[11]

The 1974 Pennsylvania highway project, with the longest stretch of prestressed concrete yet placed in this country, indicates that the method is moving out of the experimental stage and will start producing expected cost savings. A prestressed pavement was constructed near Brookhaven, Mississippi in late 1976 and to date no information on performance is available.

On the first long production run yet attempted, a slipform paver fed unbonded post-tensioning strands directly into the slabs as it placed concrete for about 1.5 miles of dual two-lane roadways.[12] The project is also the first with multiple slabs completely prestressed from one expansion joint to the next.

PRESTRESSING METHODS

There are three basic mechanical ways of applying horizontal compression to the concrete.[1] Chemical prestressing through the use of highly expansive cements has also been proposed (see Chapter 8). This chapter will only be concerned with the mechanical methods, however.

13.4 PRETENSIONED STEEL

In this method the steel strands are pulled to prescribed tension between anchors placed prior to concrete placement. After the concrete hardens and has attained a desired strength, the strands are cut near the ends and at the joints.

13.5 POST-TENSIONED STEEL

In this method, horizontal strands or bars are coated or enclosed in tubes, unstressed before concrete is cast. After concrete strength development, the steel is tensioned by jacking against the concrete end faces. In experimental pavements, steel has been placed longitudinally, longitudinally and transversely, or diagonally.

13.6 POST-STRESSED CONCRETE

This method does not use steel tendons. Plain concrete slabs are cast between anchors and compressed by jacks or wedges at ends and in transverse joints.

PRESTRESSING CONSIDERATIONS

13.7 BASIC CONSIDERATIONS

A number of basic considerations must be applied to the design and use of prestressed concrete pavements. First, the thickness and the prestressing stress must be adequate for the imposed stressed and climatic variations. Secondly, the number of joints should be reduced by making slabs as long as practically possible, consistent with economy and construction needs. Slab lengths less than 300 ft require an excessive number of joints that result in a decrease in riding comfort and a probable decrease in economy. Slab lengths of more than 700 ft, although feasible, may be critical, especially during the first period after casting, before application of the prestressing force. Finally, provision must be made at the joints to permit substantial longitudinal movement, to sustain adequate load transfer, and to protect the foundation.

Other factors to be considered when selecting slab dimensions are unfavorable profile, frequent horizontal and vertical curves, and other items that might contribute to eccentric horizontal thrust.

Pavements are but one part of the system of load-carrying ability. The other part is the subgrade. The interrelationship of subgrade and pavement has been well established by numerous tests and experience for asphaltic concrete pavements, plain, and reinforced concrete pavements. As yet, the data for prestressed pavement and subgrade interaction is relatively unknown. Theories for prestressed pavement-subgrade interaction have been postulated but have not been validated to the point of maximizing economy in design.[14] Because of its thinner section, prestressed concrete pavements are more flexible; thus, they distribute their load more efficiently over the subgrade, resulting in reduced pavement stress under wheel loads. Figure 13.1 shows longitudinal and transverse sections and indicates assumed concrete properties for a typical prestressed pavement.[1]

13.8 PRESTRESS LOSS

Because prestressing a concrete pavement increases its load capacity, the thickness of a prestressed pavement should be less than other rigid pavement types for support of equal loads. The loss of all prestress at any section of a prestressed pavement could result in a failure caused by insufficient thickness to support design traffic loads. Therefore, it is essential that a certain minimum amount of prestress be maintained at all times and at all locations of a prestressed pavement. To assure that prestress is maintained, some additional prestress should be applied initially to compensate for certain losses that will occur during and following construction. These losses could result from elastic shortening, creep, shrinkage of the concrete, relaxation in the steel, anchorage losses, tendon friction in post-tensioned systems, subgrade friction and thermal contraction. Methods for determining these losses have been developed for most prestressed members. Accepted procedures for determining such losses in prestressed pavements have been summarized elsewhere.[1,5,13,15] Table 13.1[1] gives typical slab length changes for the various effects on three slab lengths stressed to different average prestressed levels.

13.9 SUBGRADE FRICTION

With most prestress sytems, a major adverse factor is the friction between subgrade and pavement. Techniques to reduce this friction are primary considerations in construction. In most such installations, a sand layer is placed first, on which the prestressed pavement rides. A friction factor of about 1.0 is sometimes developed.[4] To reduce this a number of means have been employed between the sand and pavement slab, such as:[14]

1. A single sheet of polyethylene.
2. A bitumastic layer.
3. Double sheets of plastic between sheets of paper.
4. Double sheets of plastic with grease between.

A lower limit would appear to be about 0.5,[1,3,10] although a coefficient of 0.7 may be more realistic.[13,16] Slab length is essentially a function of prestress applied, subgrade friction restraint, and the desired minimum prestress for the midlength location. Subgrade friction is of such magnitude that it will probably be the controlling factor. At lengths greater than 500 to 600 ft, a point would be reached when the stress caused by subgrade restraint would exceed those from temperature effects and tensile stresses would be induced.[4,10] These stresses added to the load stresses would cause cracking and subsequent pavement failure.

13.10 JOINTS

Joints between the prestressed slabs probably constitute the area of most concern to prestress pavement designers. Construction joints must permit movement, must transfer shear, must be water-resistant and, most difficult of all, must respond to wheel loads about the same as the adjoining pavement, i.e., they must not be too rigid. Combinations of steel, rubber (neoprene), and concrete are utilized. The joints must be designed so that dirt, spalled concrete fragments, etc., cannot become lodged andprevent movement.[14] A durable trouble-free joint is yet to be finalized, primarily because of the lack of exposure time on a sufficient number of different types of joints to permit comprehensive evaluation. A proposed design in 1968 was not used on the Dulles installation.[1] In the past, most problems connected with the deterioration of concrete pavement have been due to the intrusion of water and undesireable pumping at joints. Apparently the joints used at Dulles (Figure 13.2) were designed to correct this problem. However, an additional modification has been made on the latest installation (Pennsylvania)[12] and further changes were made in the recent project in Mississippi.[1]

13.11 ABRASION AND CHLORIDE RESISTANCE

No information was located on the abrasion and chloride resistance of prestressed concrete pavement. However, the inherent high quality of prestressed concrete and the absence of cracking should improve the abrasive properties and chloride resistance of the exposed surface.

13.12 CRACKING AND MAINTENANCE

To be successful, prestressed concrete pavement must be essentially crack free. A small amount of transverse cracking may be tolerated,[17,18,19] but longitudinal cracking will eventually result in pavement failure. Open transverse cracks would indicate loss of prestress with impending failure due to loading of the slab or intrusion of moisture into the subbase.[20] Transient cracking in the bottom of prestressed pavements is not generally detrimental to performance.

Externally stressed pavements can buckle of "blow-up" under a combination of high stress and high temperature. This can be prevented by proper design, accurate construction, and careful adjustment of the external prestress forces as required. The incorporation of some steel in the pavement will enhance both its ultimate strength and its buckling resistance. These pavements are easily repaired; the external prestress is temporarily released and the repair accomplished by normal methods.

Internally stressed pavements cannot buckle and have usually shown greater elastic and ultimate strength than the design required.

Repairs to small areas can be readily made but major repairs may be difficult.[14] Damage to prestress pavements, especially to the reinforcing (prestressing elements) will be more detrimental and will call for more elaborate repair procedures[21] than are required for conventional pavements. The expectation is, however, that prestressed pavement will be virtually maintenance free for 40 years, while conventional concrete pavement requires heavy maintenance usually after 20 years.[12]

13.13 FATIGUE STRENGTH

Research on rigid and flexible pavements by Hveem[23] has indicated a fatigue life of 10 million applications for a 15,000-lb axle load on a 3-in. pavement if the deflection is limited to 0.02 in. maximum, with the axle in normal tracking on the pavement width (see Figure 13.3).

Under 18-kip single axle loads, the 5-in. concrete in the AASHO Road Test[24] resisted 300,000 applications before serviceability was reduced to a rating of 2.5. Under 32-kip tandem loading, 230,000 applications, respectively. It should be noted that the bases on the AASHO Road Test pumped excessively.

More recent data[25] from exploratory tests on slabs cast on foundations of different strengths indicated that an increase in the foundation strength resulted in an increase in the number of load coverages causing failure.

3.14 PRESTRESSING COSTS

While a number of analyses have been made comparing the cost of prestressed pavement with conventional pavement,[9,13,17] it is not possible, at this time, to present significant figures. This is, of course, due to the fact that a large share of the work accomplished to date in the United States has been experimental, and so the costs are naturally higher for prestressed pavement than might be expected or hoped for. It is apparent, however, that costs will be reduced when certain techniques are perfected to the point where they are more or less commonplace, and when a sufficient number of contractors are experienced enough to create competition. Reports from abroad indicate that in some countries conventional and prestressed pavement differ little in cost of construction.[3] The contractor on the Dulles project estimates that on projects of five miles or more, the unit price for prestressed pavement would approach $6.00 per square yd[9] (1973 dollars). The contractor on the Pennsylvania job believes prestress pavement could be bid equal to conventional design with experience gained on the job.[12] Contact with knowledgeable people concerned with the recent Mississippi project revealed that the substantial reduction in reinforcement (up to 90 percent) and slab thickness (25 percent) is most attractive when related to the rising cost of materials.[26] Although it is too early to define maintenance costs over a long period, it is becoming apparent that the prestressed pavement may be more economical to maintain.[12] This, however, cannot be definitely established until more service records are available.

PRECAST PRESTRESSED CONCRETE

Precast concrete is defined as concrete cast elsewhere than its final position in the structure. The systems building concept, pioneered for housing in Europe after World War II, added appreciably to the precast concrete field. By far the greater portion of precast concrete used to date has been either for vertical construction or fabrication of structural components. The process has come into widespread use within the past 10 years with ample documentation of design procedures,[27] handling and erection methods,[28] and specialized casting techniques.[29]

No evidence was found where pure precasting has been applied to pavement construction. The South Dakota Department of Highways installed a series of 6-ft x 24-ft x 4-1/2-in. prestressed precast panels in a highway test section in 1968,[30] and covered the panels with asphaltic concrete after installation. Advantages of this system over prestressed pavement were said to be elimination of subgrade friction and reduction of shrinkage cracks. On-site precasting of roadway elements would offer certain advantages. Assembly line casting, handling, and placing of identical units could be economically attractive. Twenty-four hour production would offer full use of equipment. Production could proceed during inclement weather. Proximity to the jobsite should make for more efficient operations and more effective use of personnel. Apparently the South Dakota installation proved successful although service records to date, if available, would be for only short duration.

There are problems which may preclude the use of precast sections for roadways. The amount of steel required in the design of the section may not be sufficient for handling and erecting. It would appear to be uneconomical to add additional steel primarily to get the section in place. The area in precast construction where most of the trouble develops is, of course, the joint, and there would be many joints in a precasting pavement installation (220 per mile at 24 ft length). Connections in precast construction are complete joints; i.e., they separate two entirely separate elements and thus could be expected to have all of the problems associated therewith. Joint maintenance undoubtedly would cause much concern in precast pavement construction.

Due to the fact that precast prestressed pavement has been used very sparingly to date and the obvious problem of attending to many joints over the life of the roadway, further study of this type of pavement construction is not recommended.

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Table 13.1

Computed Slab Length Changes[1]

Slab Length, ft	400	600	900
Average prestress, psi	225	325	420
Progressive changes:			
Shrinkage, in.	0.2	0.3	0.45
Elastic shortening, in.	0.36	0.78	1.51
Creep, in.	1.1	2.3	4.55
Total shortening, in.	1.66	3.38	6.51
Cylical changes (mature Concrete, E = 5 x 10^6 psi):			
Seasonal change, in.	1.15	1.73	2.59
Daily movement	±0.20	±0.27	±0.32
Total maximum cyclic change, in.	1.55	2.27	3.23
Change at each end, in.	0.8	1.15	1.6

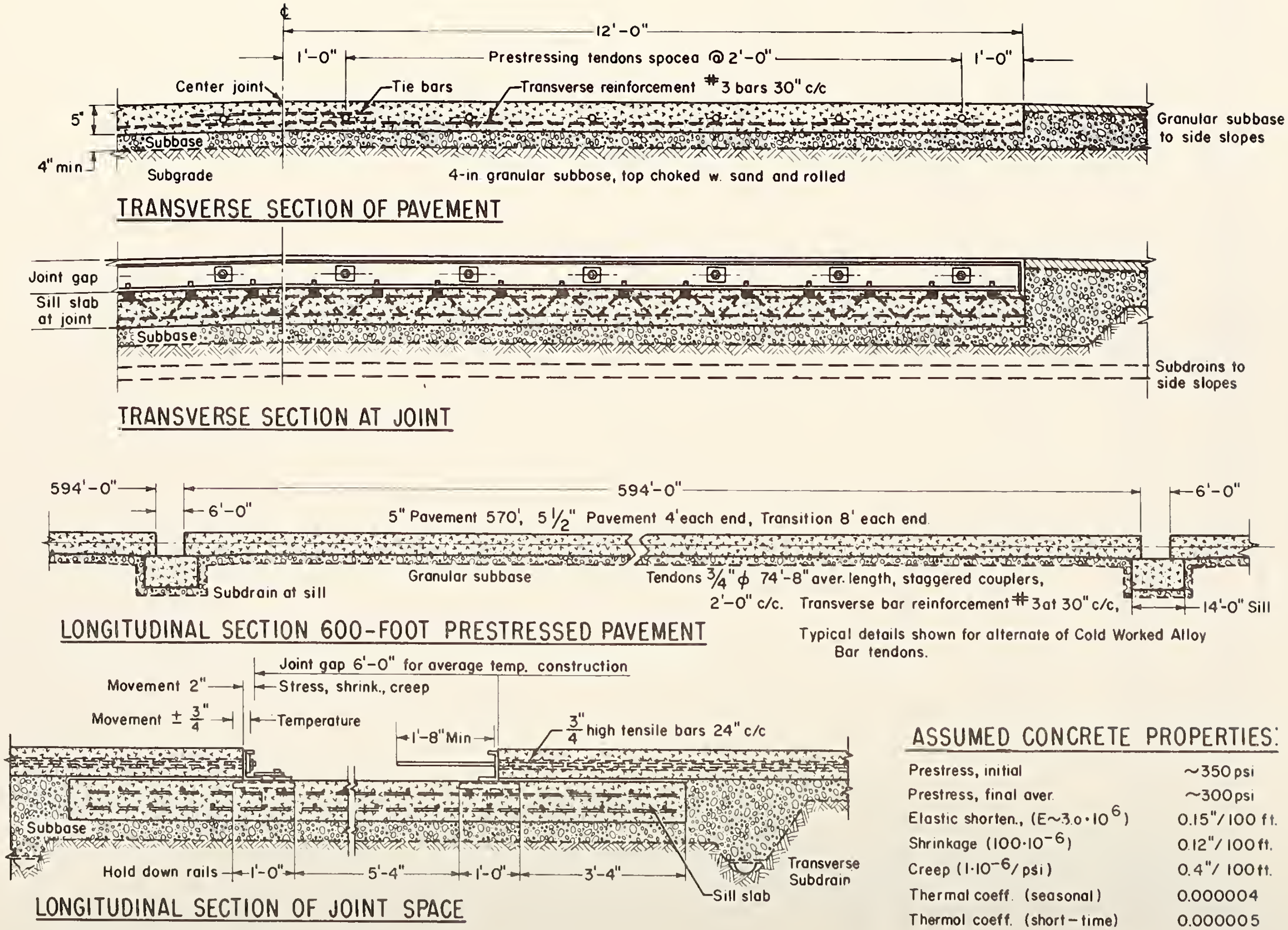

Figure 13.1. Longitudinal and transverse sections of proposed prestressed pavement[1]

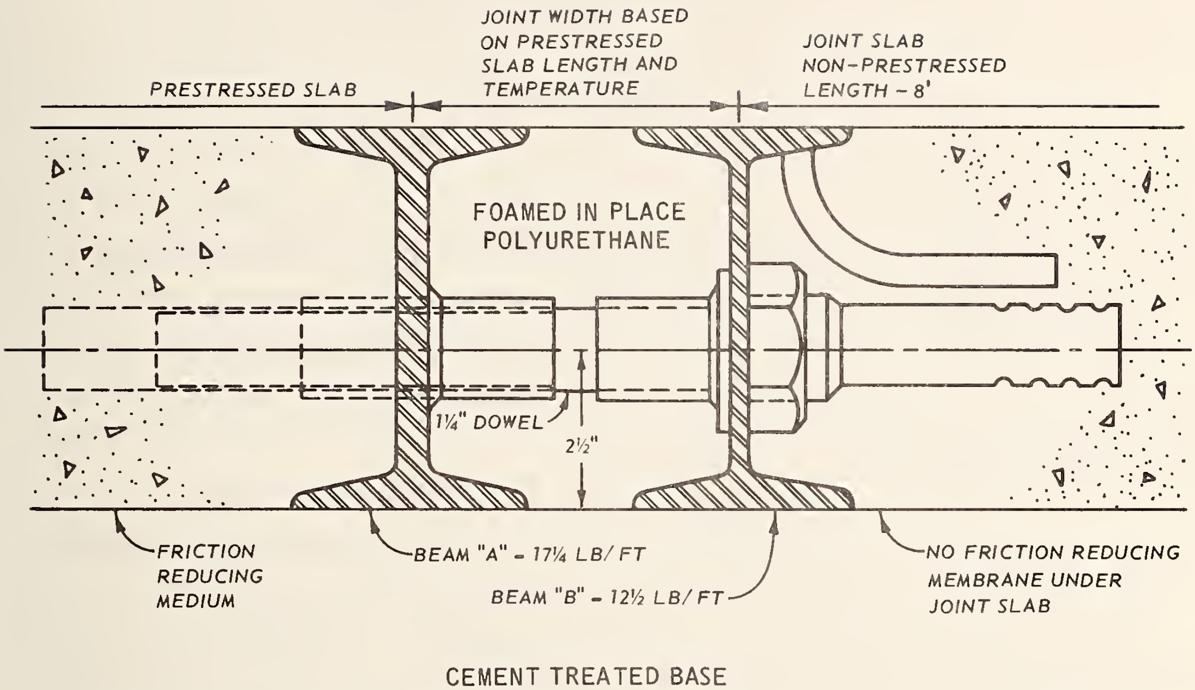

Figure 13.2. Joint detail of longitudinal section--Dulles Airport

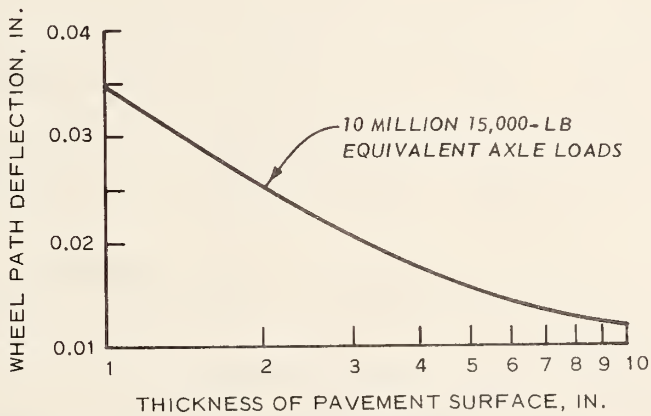

Figure 13.3. Suggested relationship between pavement thickness and deflection based on California pavement performances on stable bases and subgrades (15,000 lb mean single axles)[23]

CHAPTER 14

VACUUM-PROCESSED CONCRETE

INTRODUCTION

14.1 BACKGROUND

Vacuum-processed concrete (VPC) is portland cement concrete, mixed and placed in the usual manner, but immediately subjected to a vacuum applied to one or more surfaces through vacuum chambers in contact with the unhardened concrete thus resulting in removal of a significant portion of the mix water.

The vacuum process was invented in 1935 by Karl P. Billner. Immediately before and after World War II, the process was tried on a wide variety of concrete structures both in the U.S. and Europe for mass structures,[1,2] canals and structural components,[3] floors and walls for housing,[4] and bridge decks.[5] Due primarily to the cost, bulkiness of the equipment, and inappropriate application in some cases, the process did not gain wide acceptance. After a period of decline, the vacuum process for finishing concrete is making a comeback in Northern Europe. Over 5 million square feet of VPC surfaces were produced in the Scandanavian countries in 1971 and the annual production of it is increasing rapidly.[5]

The recent resurgence of application of VPC is believed to be due to:

a. The devlopment of effective, easy-to-use and inexpensive processing equipment.

b. A cooperative attitude by labor.

c. Concentration of application for which the VPC is best suited, i.e., industrial, warehouse and apartment floors, structural slabs, and parking decks.

14.2 DESCRIPTION OF SYSTEM

The procedure for vacuum processing is rather straightforward. First, the concrete is placed in the forms in the usual manner. Since fresh concrete contains a continuous system of water-filled channels, the application of a vacuum to the surface of the concrete results in a large amount of water being extracted from a certain depth of the concrete. In other words, what might be termed "water of workability" or "water of convenience" is removed when no longer needed. The vacuum upsets the equilibrium originally existing between the external atmospheric pressure and the hydrostatic pressure of the overlying concrete on one hand, and the internal intergranular and interstitial pressure on the other. The free water is squeezed out and is removed through the vacuum chamber while the solids consolidate and densify. The final water/cement ratio is thus reduced and the physical properties improved.

The plant required consists of a filtering mat, which will enable water to be drawn off without the fines from the concrete, a vacuum pump, and a separator to prevent the water drawn off from passing through the pump (Figure 14.1).

As shown in Figure 14.1, the filtering mat consists of a flexible or rigid watertight backing piece A with a seal B around its periphery. Within the space formed by this rubber seal, a sheet of expanded metal (C) is placed first and is followed by a sheet of wire gauge (D). The gauge and seal are then covered by a sheet of linen or muslin which acts as a filter and permits the water to be drawn out without any cement. The guage and expanded metal form a space between the linen and backing board A in which the vacuum can be maintained and the water drawn off. This vacuum reduces the water content by up to 20 percent over a depth of 150 to 300 mm (6 to 12 in.). The reduction is greater nearer to the mat and it is usual to assume the suction to be fully effective over a depth of 150 mm (6 in.). Thus the process would appear to be especially beneficial for slab or pavement construction. The sequence of operations in the vacuum concrete process is given in Figure 14.2.

The amount of water extracted from fresh concrete and hence the subsequent reduction in water/cement ratio is dependent on several parameters. Two of the most significant are the initial water/cement ratio and the time of processing (vacuuming). Orchard[7] has shown that the greater the initial water/cement ratio the larger the possible water reduction. In Figure 14.3 and Table 14.1, 0.80 water/cement ratio concrete was reduced an average of 0.22 after 25 minutes of processing; 0.65 was reduced an average of 0.20, but 0.50 concrete was reduced only 0.11. Significantly also, approximately 75 to 80 percent of the reduction is accomplished in the first 10 minutes of processing.

MECHANICAL PROPERTIES

14.3 COMPRESSIVE STRENGTH

The reduction in water/cement ratio achieved by vacuuming has a beneficial effect on almost all properties of the hardened concrete. Typical of the increase in compressive strength is the work conducted at Jydsk Technical Institute, Denmark,[8] and presented in Figure 14.4. Apparently a strength increase of about 40 percent may normally be expected when processing conventional concrete. If vibration can be incorporated in the vacuum processing, additional water can be removed and higher strength achieved. Typical is Garnett's work[9] given in Table 4.2. However, removal of more than about 20 percent of the mix water appears to be unwarranted and may even prove detrimental as indicated in Figure 14.5.

14.4 FLEXURAL STRENGTH

Although the lowering of the water/cement ratio by vacuum processing might be expected to increase the tensile and flexural strength of the concrete to the same degree as the compressive strength, this does not appear to be the case. Cross[11] reports that flexural strength is improved only about 25 percent. Due to the limited information available, however, this observation should be viewed with caution.

14.5 TENSILE STRENGTH

The work of Bentz, et al.,[10] (Table 14.3) indicates that the indirect tensile strength is increased also by approximately 25 percent by vacuum processing. Again, due to the limited information available, these indications should be treated with caution.

14.6 MODULUS OF ELASTICITY

Limited data on the modulus of elasticity of VPC was located.[12] Figure 14.6[12] indicates that the modulus is affected by vacuum processing (essentially by increases in the compressive strength with no changes in ultimate strain) with increases of 30 to 40 percent over nontreated concrete. No explanation was given for the difference in modulus between 0.6 and 0.5 water/cement ratio for both VPC and nontreated concrete.

14.7 DIMENSIONAL CHANGES

Most of the cracks associated with rebar corrosion and spalling result from drying shrinkage of the concrete rather than thermal volume change or line load stress.[15] It is well known that the drying shrinkage and permeability of concrete are closely related to the unit water content, i.e., the amount of water per unit volume at the time of setting. Thus the reduced water content accomplished by vacuum processing should effect a comparable reduction in shrinkage. Reductions of 30 percent[5] to 50 percent[16] in drying shrinkage for VPC have been reported. More specifically, Brux[12] presented data for early age shrinkage on conventional and vacuum processed concrete (Figure 14.7). Although no reference was located relative to the creep characteristics of VPC, rationale would indicate that the creep parameter (basic and drying creep) would be reduced proportionally to the water reduction achieved with vacuum processing.

14.8 DURABILITY

The vacuum process removes not only the excess water but also the air bubbles from the surface of the concrete since they do not form a continuous system.[6] It can therefore be used with air-entrained concrete Leviant[13] has shown that vacuum-treated air-entrained concrete has significantly higher frost resistance than nonair-entrained vacuum concrete or air-entrained conventional concrete. The method of evaluation was reduction of elastic moduli. Hope and Quelch[14] have shown that the compressive strength and pulse velocity of specimens frozen at early ages are greater for vacuum-processed specimen than non-processed concrete (Figure 14.8). The concrete mixture used for the work reported in Figure 14.8 has a water/cement ratio of 0.71, a slump of 2 in. an air content of 3.7 percent, and a nominal 28-day strength at 3000 psi.

The Chalmers University of Technology group in Sweden investigated the performance of parking decks subject to repeated cycles of freezing and thawing and deicing salt application. About 80 percent of all decks investigated showed cracking through the slabs sufficient to let water through. None of the decks which had been vacuum treated showed any failures, and the report ends with the recommendation that all parking decks should be vacuum treated. This recommendation is followed today.[8]

The Federal Highway Administration investigated the chloride permeability and the resistance to deicer sealing under freeze-thaw conditions of VPC.[19] After more than 330 salt applications to the 0.50 initial water/cement ratio, the average chloride content of the vacuum-treated slabs at a 1-in. depth was 1.2 lb/yd^3 whereas, in the untreated slabs, 3.6 lb/yd^3 were found. Vacuum treatment thus was effective in reducing deicer penetration, although it did not prevent penetration for an unlimited time. Laboratory freeze-thaw tests conducted according to ASTM C 672 showed that the vacuum treated concrete was extremely resistant to deicer scaling. Results are given in Table 14.4.

14.9 ABRASION RESISTANCE

VPC is apparently gaining acceptance in the United Kingdom as a technique to improve wear resistance of concrete floors.[17] Paulsson[18] has suggested that concrete surfaces treated with vacuum water experience 16 to 39 percent less wear than traditionally laid floors. Cross[11] claims that vacuum treatment increases the abrasion resistance of concrete surfaces by up to 300 percent. Part of this increase is due to the reduction in water/cement ratio caused by the removal of water. In addition, suggestion[6,11] is made that the pressure exerted on the surface closes the voids and eliminates splitting, thus resulting in a surface highly resistant to abrasion.

14.10 TOUGHNESS

No information on the toughness of VPC could be located.

14.11 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

No information on the fatigue strength or dynamic response of VPC could be located.

14.12 PERMEABILITY

The permeability of VPC should be improved over that of conventional concrete because of the densification of surface of the concrete resulting from the removal of excess void producing water.

USE OF VACUUM PROCESSED CONCRETE

14.13 EASE OF CONSTRUCTION

VPC does involve the addition of another process and more equipment to the construction operation. However, for highway construction the additional effort and cost would appear to be partially overcome by the advantages. For example, the vacuum process now used in Europe employing larger, yet lighter mats with more efficient pumps, is most effective when used on large unobstructed areas, such as warehouse floors or highways. In the vacuum process, per se, as the water is removed and the solids consolidate, a very rapid thixotropic hardening occurs. In a matter of minutes the plastic concrete becomes sulf-supporting with a comparative strength of about 20 psi.[5] Thus a time savings may be realized by reducing the waiting time between placing and finishing. The problem of variation in slump from batch to batch is of nominal consequence when vacuum dewatering is used, because the vacuuming evens out variations in water content. The system is not complicated. The mats in use today, if damaged, can usually be repaired in the field. The average life of vacuum mats with normal use is claimed to be eight months or more.[5] Repairs on the piston-type pump can be made by any compressor mechanic.

Vacuum dewatering of concrete is especially valuable for winter work where job savings in heating costs and protection can be realized. It is also valuable during periods of unsettled weather or in moist climates because outdoor slab construction can continue through mist or light rain without damage to the work.[20] There is never a large area of unfinished concrete exposed, and freshly placed concrete can be quickly finished and protected.

The vacuum concrete process should be compatible with the conventional paving operation. The sequence of operations shown in Figure 14.2 could follow a paving train with minimal changes in the vibration arrangement. Normal mixture proportions can generally be used without any changes. Occasionally a condition known as "crossing" occurs in which a layer of fines forms at the surface and restricts the flow of water through the mat. It is practically always possible to resolve the problem by reducing the sand-aggregate ratio in the mixture.[20,21] Finishing and smoothness requirements should present no problem although power troweling is required.[5,20] Reports are that good crews can produce floors of Class 1 quality, the highest quality required in Sweden, with vacuum processing.[22]

14.14 EASE OF MAINTENANCE

The difficulty of performing maintenance, if ever needed, on a VPC pavement would not be any different than for a conventional concrete pavement.

14.15 ENVIRONMENTAL COMPATIBILITY

The environmental compatibility of VPC should be on the same level as any conventional concrete pavement.

14.16 AVAILABILITY AND COST

Almost all vacuum processing equipment in use today is made by three companies on their licenses. AB Skanska Cementgjuteriet of

Stockholm, Sweden, has been producing and using vacuum processing equipment since 1956. Primarily a contractor rather than an equipment manufacturer, Skanska produces only enough equipment for its own needs. Hoff and Company A-S, of Copenhagen, Denmark, has been producing and selling vacuum-processing equipment since 1965. Tremix AB, of Nacks, Sweden, introduced a line of vacuum processing equipment in 1968. At the present time (Aug 1975), the availability of vacuum processing equipment in the United States is somewhat limited. The Tremix and Hoff systems are available through two outlets in the United States.[23]

The cost of the equipment, which consists of a vacuum pump with electric drive and two suction mats of 325 sq ft each, is about $6000 to $7000.[20] Operating costs are small, consisting of electricity for the motor of the pumps and occasional replacement of the filter pads. Labor cost is also small. The reduced amount of labor may be a possible obstacle to acceptance of VPC in the United States because of opposition by organized labor. Since finishing is by plant equipment immediately after resumming, it has been suggested that the number of cement finishers may be reduced as much as 50 percent.[22] The actual cost of the use of VPC would require consideration of all factors such as finishing time saved and reduction in cement requirement through achievement of lower water/cement ratio.

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CHAPTER 14

VACUUM PROCESSED CONCRETE

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Table 14.1

The Rate of Extraction of Water with Different Concrete Mixes[7]

Mix No.	Degree of Vacuum of Mercury in.	Maximum Aggregate Size in.	Initial Water/ Cement Ratio	Average Reduction of Water/Cement Ratio Determined by Weight of Water Extracted	Processing Depth in.
1	24	3/4	0.80	0.27	4
2	28	3/4	0.80	0.17	12
3	28	3/4	0.65	0.19	4
4	--	3/4	0.50	0.10	4
5	20	3/4	0.80	0.23	4
6	30	3/8	0.80	0.24	4
7	28	1-1/2	0.65	0.21	4

Table 14.2

Water/Cement Ratio and Strength of Vacuum-Processed Concrete

Water/Cement Ratio		Compressive Strength	
Before Processing	After Processing	Before Processing psi	After Processing psi
0.74	0.68	2,570	3,330
0.71	0.59	2,180	3,230
0.65	0.57	2,990	3,920
0.60	0.55	4,300	4,760

Note: Twenty minutes of processing accompanied by vibration between 4th and 8th and 14th and 18th minutes.

Table 14.3

Compressive and Indirect Tensile Strength of Industrial Concrete Floors[10]

Concrete Mix	Treatment*	Specimens	Compressive Strength (psi) 3 Days	Compressive Strength (psi) 28 Days	Indirect Tensile Strength (psi) 3 Days	Indirect Tensile Strength (psi) 28 Days
Floor Part A	a, b, c	3-3/4-in. cores	3890	5800	465	580
	d	6-in. standard cylinders (comp), 4-in. cylinders tension	2120	3840	335	495
Floor Part B	a, b, c	3-3/4-in. cores	3920	5950	480	610
	d	6-in. standard cylinders (comp), 4-in. cylinders tension	2540	4340	375	510
Floor Part C	e	3-3/4-in. cores	3050	5020	335	493

* a - Vacuum dewatered; b - power float finish; c - air cured; d - fog cured; and e - air cured (no processing).

Table 14.4

Scaling Resistance of Various Water/Cement Ratio Concrete

Mixture parameters (b)	Scaling rating at 50 cycles (b)	Scaling rating at 100 cycles (b)
1. W/C = 0.40, CF = 7.0	1	2
	2	3
	1	2
W/C = 0.50, CF = 7.0	2	2
	2	2
	2	2
2. Vacuum processed*		
a. W/C = 0.40, CF - 7.0	1	1
	0	0
	0	0
b. W/C - 0.50, CF - 7.0	1	1
	0	1
	0	1

Note: (a) 14" x 10" x 3" specimens tested in accordance with ASTM c 672-72 T. Three specimens for each water/cement ratio concrete.

(b) Visual scaling rating based on the 0 to 10 severity scale below:

0 - no scale
1 - scattered spots of very light scale
2 - scattered spots of light scale with mortar surface above corase aggregate removed
3 - light scale over about one-half of the surface
4 - light scale over most of the surface
5 - light scale over most of the surface, with a few moderately deep spots, where the mortar surface was below the upper surface of the coarse aggregate
6 - scattered spots of moderately deep scale
7 - moderately deep scale over one-half of the surface
8 - mmoderately deep scale over the entire surface
9 - scattered spots of deep scale with the mortar surface well below the upper surface of the coarse aggregate; otherwise moderately deep scale
10 - deep scale over the entire surface

* 236 grams of water removed in 7 minutes for the W/C=0.40 slabs and 409 grams of water removed in 7 minutes for the slabs with a water-cement ratio of 0.50.

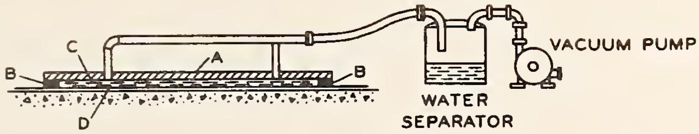


Figure 14.1. General arrangement for vacuum concrete process

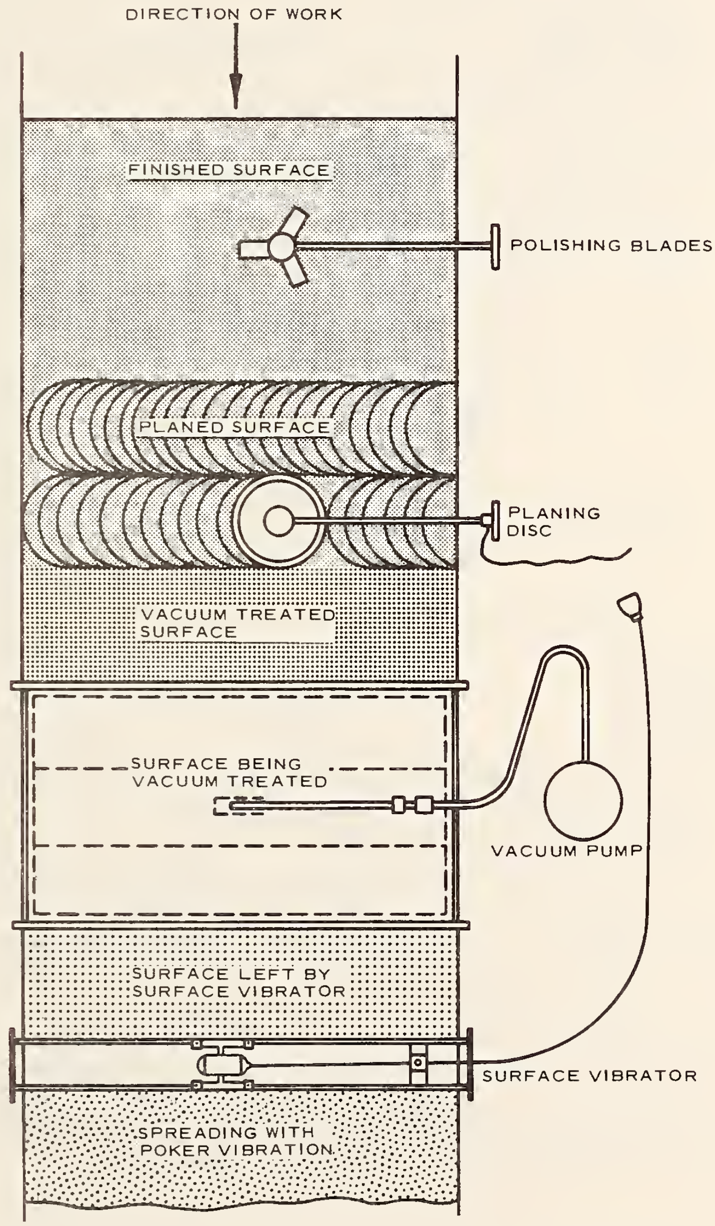


Figure 14.2. Sequence of vacuum processing operations for concrete[5]

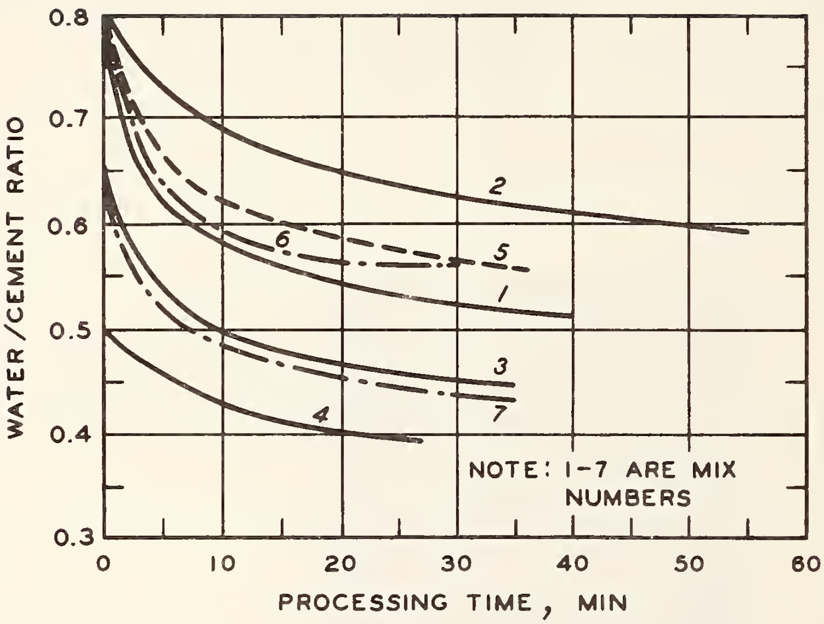


Figure 14.3. Rate of depression of water/cement ratio with vacuum processing time for top processing

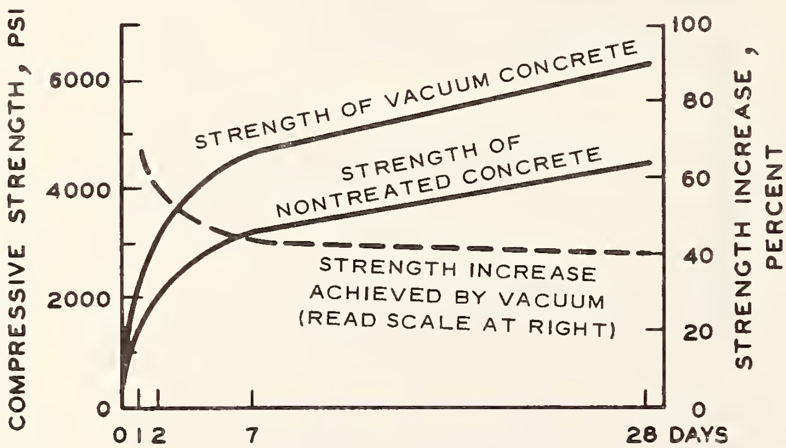


Figure 14.4. Comparison of compressive strength of vacuum concrete and nontreated concrete[8]

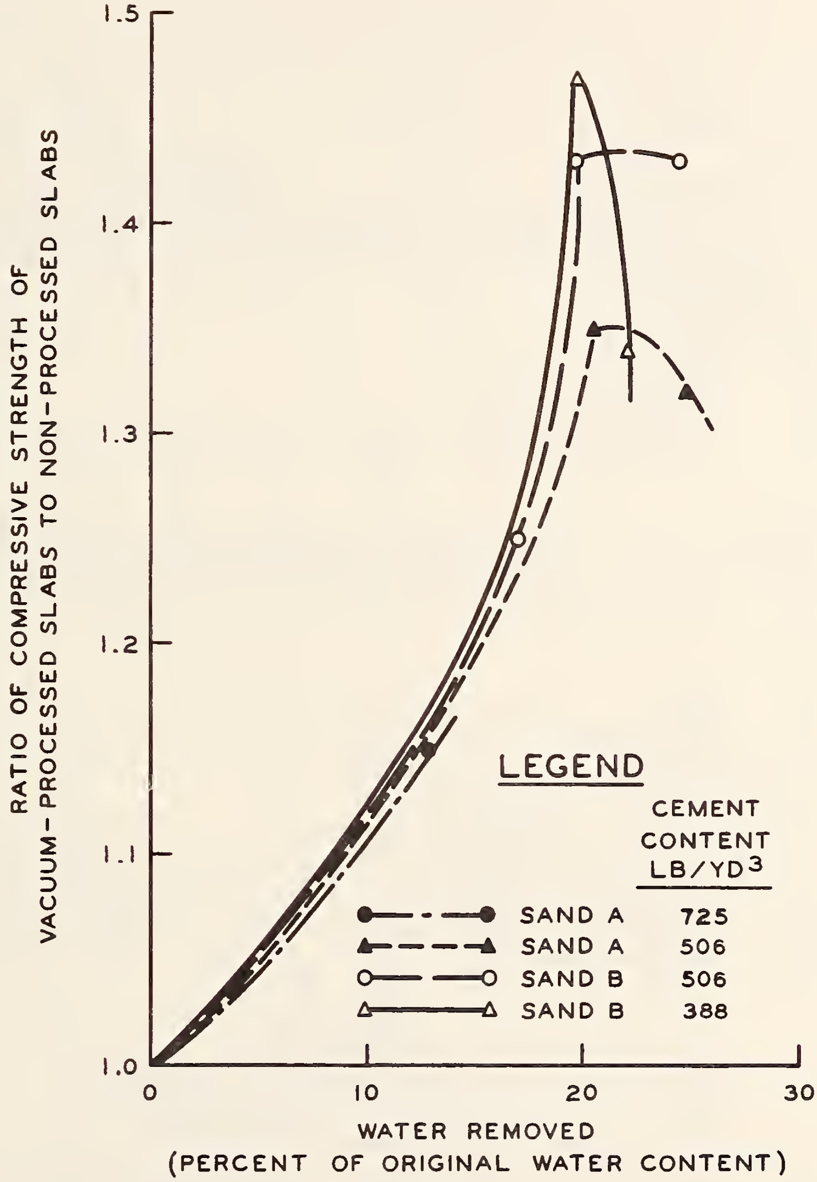


Figure 14.5. Ratio of compressive strength of vacuum processed slabs to nonprocessed slabs versus percentage of water removed[10]

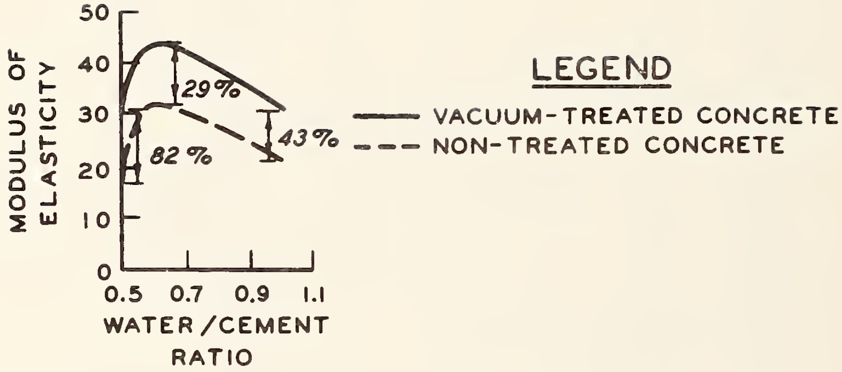


Figure 14.6. Variation in the modulus of elasticity with the water-cement ratio at the time of mixing of the vacuum-treated and nontreated concrete[12]

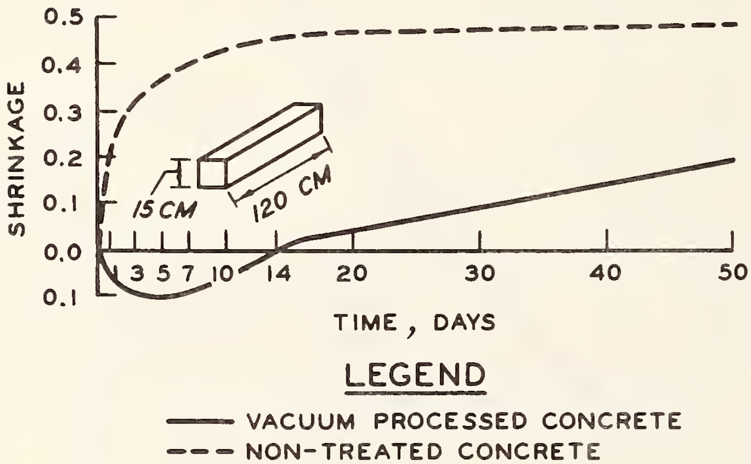
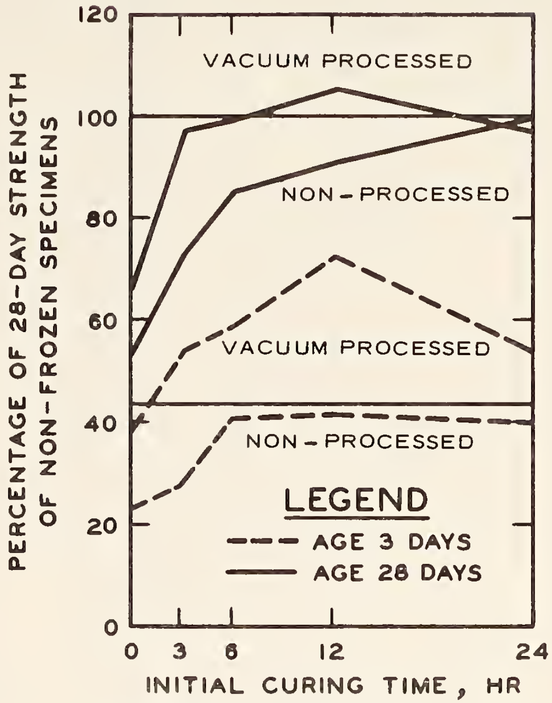
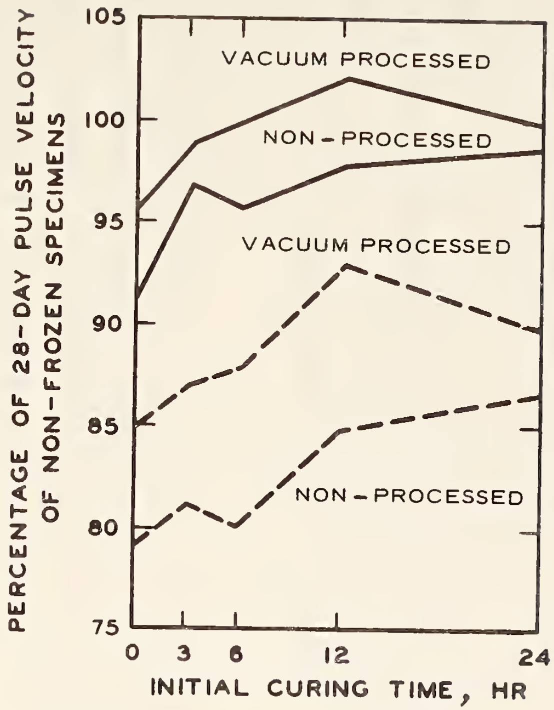


Figure 14.7. Shrinkage of vacuum-treated and nontreated concrete with time[12]



a. COMPRESSIVE STRENGTH OF FROZEN SPECIMENS



b. PULSE VELOCITY OF FROZEN SPECIMENS

Figure 14.8. Compressive strength and pulse velocity of frozen specimens of vacuum-treated and nontreated concrete[14]

CHAPTER 15

CONCLUSIONS AND RECOMMENDATIONS

SUMMARY OF FINDINGS

The amount and detail of the information contained in the previous chapters vary for each material or system because some are relatively new or not popular and little has been written about them, while others have had either extensive use and documentation or limited use but with extensive promotion. Task D of this study was to rate each material or materials system and recommend future development and refinement work on these materials. Rating is somewhat difficult for all the materials or materials systems, because each may have different characteristics which contribute to the concept of having a zero maintenance material. The following sections briefly summarize the findings for each material in a manner which allows the reader to get some indication or "rating" as to how each of these materials or materials systems compares to either asphalt concrete or portland cement concrete. Conclusions and recommendations follow.

15.1 ASPHALT CONCRETE

Conventional asphalt concrete is a high quality thoroughly controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass. The following is a summary of some of the characteristics of this material:

Temperature Dependency:

Stress-strain characteristics are temperature dependent. Changes in test temperature can produce large changes in strength characteristics.

Compressive Strength:

Limited data. Values between 35 to 285 psi (unconfined) reported. Changes in asphalt type and content produce values still within this range. More influenced by temperature and rate of loading than other factors.

Marshall Stability:

Typical values range from 200 to 2000 lb with associated flows at 0.01 in. of 8 to 22. Values to 4000 lb are possible, however. Stability values and flow increase with increasing asphalt content up to an optimum and then decrease with increasing asphalt content. Decreases in asphalt penetration increase stability. Affected by aggregate type and size. Improved when using crushed aggregates instead of ungraded natural aggregates and when using crushed fine material instead of crushed coarse material. Increased angularity of aggregate also improves stability. Affected significantly by temperature changing from 4000 lb at 86F to 400 lb at 176F.

Flexural Strength:

Values from 4 to 1200 psi reported. Affected by temperature, rate of loading, and penetration grade of asphalt. Lower penetration grades produce better strengths. Not significantly influenced by asphalt content and aggregate variations.

Tensile Strength:

Values from 13 to 775 psi reported. Extremely sensitive to temperature changes, with values changing by an order of magnitude for 60F temperature changes. Less sensitive to load rates, but faster loading gives better results. Not significantly affected by changes in asphalt content or by aggregate variations.

Dimensional Changes:

Exhibits two different thermal coefficients of expansion and contraction, one below the transition temperature of asphalt (solid thermal coefficient) and one above (fluid thermal coefficient). Values of 1.32 to 1.63 per degree F x 10^{-5} have been reported. Influenced by source of asphalt and penetration grade. When made with absorptive aggregates, can swell and shrink with moisture changes.

Durability:

Data not available.

Abrasion Resistance:

Data not available.

Toughness:

Data not available.

Fatigue Strength and Dynamic Conditions:

Influenced by binder content, void content, penetration grade, aggregate type, temperature, and whether the tests are conducted as controlled stress (load) or controlled strain (deflection). In general, a decrease in penetration produces an increase in fatigue life for controlled stress mode and a decrease for controlled strain mode. The same behavior occurs when other variables are changed such as aggregate roughness, increasing angularity, change in aggregate gradations from open to dense or temperatures being decreased. Increases in asphalt content increase fatigue life in both modes as does decreases in air void content. In general, asphalt concrete is rate sensitive with higher strength values obtained for faster rates of load or strain.

Permeability:

Directly affected by air void content, decreasing with decreasing void content. Essentially impermeable to air and water at void contents less than 6 percent.

Ease of Construction:

Widely used. Equipment and techniques for construction well developed. No special problems anticipated.

Ease of Maintenance:

Requires continual maintenance. Repairs are very simple to perform but are usually not performed adequately.

Availability and Cost:

Availability usually not a problem except in times of petroleum product shortages. Costs continually increasing at rapid rates. F.O.B. costs of asphalt cement in 1975 were approximately $64/ton and have increased since then. In-place costs of asphalt concrete depend on local labor conditions, shipping costs, pavement thickness, and many other factors.

15.2 GUSSASPHALT

Gussasphalt is a mixture of a relatively hard asphalt, mineral filler, aggregate, and sometimes Trinidad Lake Asphalt. These materials are proportioned to produce an essentially voidless mixture which can be cast or poured in place at temperatures of 374 to 437F. The following is a summary of the characteristics of this material:

Compressive Strength:

Generally dependent on temperature. Strengths are generally less than conventional bituminous materials at temperatures between 30 and 75F but are greater when temperatures exceed 75F. Strengths from 300 to 1400 psi are possible. Effects of binders, fillers, and curing are not known.

Flexural Strength:

Data not available.

Tensile Strength:

Generally dependent on strain rate and temperature. At low rates of strain (0.004 in./min) and temperatures less than 60F, tensile strengths (100 to 300 psi) are less than conventional bituminous materials. At faster strain rates (0.04 in./min), strengths increase (150 to 400 psi) and are better than conventional bituminous materials at 50F or greater. For even faster strain rates (0.4 in./min), tensile strengths are better (200 to 500 psi) than conventional bituminous materials at all temperatures.

Dimensional Changes:

Data not available.

Durability:

Good performance in actual pavements for 15 years.

Abrasion Resistance:

Wear resistance is temperature dependent and improves with decreasing temperature. At less than 14F, tends to chip and break. Independent of aggregate types and preliminary treatments.

Toughness:

Data not available.

Fatigue Strength and Dynamic Conditions:

Data on fatigue not available. Tensile results indicate strain rate sensitivity when loaded.

Permeability:

Permeability is generally low.

Ease of Construction:

Very similar to portland cement concrete construction. Placed at 400F thus causing some safety problems.

Ease of Maintenance:

No special problems exist.

Availability and Cost:

Limited availability in the United States. Costs are two to three times that of conventional asphaltic concrete.

15.3 ASBESTOS ASPHALTS

Asbestos asphalt is simply an asphalt mixture which has asbestos fibers added to it. There are several different grades or groups of asbestos fibers, but the asbestos that is generally used in pavement construction is a short fiber type such as a 7M type. Asbestos to asphalt ratios vary from 0.3(min.) to 3 (max) percent. The following is a summary of the characteristics of this material.

Compressive Strength:

Temperature sensitive with strength increases occurring for decreases in temperature. Reacts to changes in binder content the same way a conventional asphalt mixture does. Fiber additions increase strength to values of 650 to 1000 psi. Very sensitive to compactive effort with strength reduced by 70 to 80 percent for a 3 to 4 percent reduction in density.

Marshall Stability:

Similar behavior to conventional asphalt mixtures. Asbestos additions allow increased binder contents (1 to 3 percent) with no reduction in stability value. Aggregate type does affect stability. Asbestos content increases generally increase stability value.

Flexural Strength:

Data scarce. Asbestos additions to asphalt concrete do not significantly increase flexural strength and may decrease it. Values of 190 to 325 psi reported. Temperature sensitive in the same manner as compressive strength.

Tensile Strength:

Values of 135 to 235 psi at room temperatures reported. Temperature sensitive with increasing strengths (to 600 psi) at decreasing temperatures (20F). Zero strength at 120F. Increased binder content increases strength. Affected by type and amount of filler. Fiber additions improve strength.

Dimensional Changes:

Data not available.

Durability:

Six years of service in severe freeze and thaw environment shows excellent performance. Asbestos additions reduce rutting at high temperatures and are more independent of traffic volumes to produce rutting. At 130F asbestos asphalts perform like nonasbestos asphalt concrete.

Abrasion Resistance:

Data not available.

Toughness:

Data not available, but fiber additions should improve it.

Fatigue Strength and Dynamic Conditions:

Data not available but fiber additions should improve fatigue resistance.

Permeability;

Data limited. Permeability dependent on amount of compactive effort. Standard laboratory densities are impermeable. When permeable, fiber type affects the magnitude of the permeability.

Ease of Construction:

Can be placed using the same methods as conventional asphalt concrete but will require more compactive effort.

Ease of Maintenance:

No more difficult than for conventional asphalt pavements.

Environmental Compatibility:

Potential health hazard when working with loose asbestos fibers but precautionary measures are available.

Availability and Cost:

Availability is not a problem. Cost will be between 10 and 20 percent greater than for conventional asphalt mixtures because of increased asphalt contents and the addition of the asbestos fibers.

15.4 SULFUR MODIFIED ASPHALT

At the processing temperatures of asphalt, sulfur becomes a liquid and can be combined in the asphalt concrete. After cooling, the sulfur solidifies in the voids of the asphalt concrete mixture and acts as a filler. The following is a summary of the characteristics of this material.

Compressive Strength:

Data not available.

Marshall Stability:

Stability is increased with sulfur additions to 2 to 4 times that of conventional additions (4 to 16 percent). Values range from approximately 200 lb at 4 percent to from 2000 to 4000 lb at 16 percent depending on aggregate type. Increasing asphalt content for a given sulfur content tends to reduce stability, but the rate of decrease is not as rapid as for conventional asphalt mixtures. Sulfur content should be at least equal to the asphalt content to improve stability.

Flexural Strength:

Limited data. Apparently strain rate and temperature sensitive. Values ranged from 46 to 415 psi.

Tensile Strength:

Temperature sensitive. Strengths less than 100 psi at 68F but at 50F are 2 to 3 times greater. Strength increases with increase in sulfur content and decrease in asphalt content. Affected by surface angularity of aggregates.

Dimensional Changes:

Data not available.

Durability:

Data not available.

Abrasion Resistance:

Limited data. Performance at early ages appears good.

Toughness:

Data not available.

Fatigue Strength and Dynamic Conditions:

Mixed performance on fatigue tests indicating both lower fatigue life and greater fatigue life than that of conventional asphalt concrete. Differences may be the result of differences in mixture compositions. Data on dynamic load effects not available.

Ease of Construction:

Slightly more involved than using conventional asphalt mixtures as special equipment for storing, heating, and mixing sulfur will be required. Can be adapted to batch hot-mix plants, however.

Ease of Maintenance:

Similar to that of conventional asphalt concrete.

Environmental Compatibility:

Problem exists with hydrogen-sulfide fume concentrations. Fumes cannot be eliminated, but levels can be controlled.

Availability and Cost:

Availability is good. The cost of sulfur is slightly less than the cost of asphalt cement. Any reasonable additions of sulfur to asphalt will increase total material costs from 10 to 20 percent.

15.5 NONCALCAREOUS INORGANIC CEMENTS

Noncalcareous inorganic cements include sulfur, phosphate cements, sorel cement (magnesium oxychloride cement), sodium silicate, the chlorides, and others. Only sulfur and phosphate cement were found to have any promise for pavement applications. Sulfur can be used in its elemental form or it can be modified. It can be used to (1) impregnate or infiltrate concrete, (2) as a coating, (3) as the principal binder, (4) as an admixture, and (5) as a foam. Phosphate cements are a proprietary product which can be used as either cold setting or hot setting materials. The following is a summary of the characteristics of these materials.

Sulfur-Impregnated or Infiltrated Concrete:

Strength, modulus of elasticity, and resistance to freezing and thawing all increase with sulfur impregnation which uses a vacuum process. Sulfur contents from 8 to 13 percent produce strength increases from 3 to 10 times that of the basic concrete. No long-time service records exist.

Sulfur-Surface Applications:

Similar to sulfur impregnation but no vacuum is used. Improvements are also similar but not quite as pronounced.

Sulfur Admixture:

Limited data. Additions of 2 percent powdered sulfur to cements and mortars which were then autoclaved produced compressive strength increases in the paste and decreases in the mortar. Increased drying shrinkage. Decreased density and proosity. Increased microhardness.

Sulfur Concrete:

Strength gain of sulfur concrete is considerably more rapid than portland cement concrete but end result is similar. Creep is several times greater than that of conventional concrete. Modulus of rupture and modulus of elasticity similar to conventional concrete. Poor freeze and thaw resistance.

Sulfur Foam:

Strength related to density and increases with increasing density. Freeze and thaw resistance is satisfactory.

Phosphate Cements:

Strengths of 5000+ psi possible in one hour. Other properties were equal to or better than those of portland cement concrete.

15.5.1 Sulfur.

Ease of Construction:

Sulfur-impregnation and infiltration processes are suitable for precast concrete elements and existing bridge decks and pavements. Techniques to do this are not yet perfected, however. Sulfur-surface applications can be used anywhere by simply brushing or spraying molten sulfur on any desired surface. Sulfur-admixture processes will be very difficult to implement in the field. Sulfur concrete production should provide no serious difficulties but equipment and techniques for large productions must still be developed. Sulfur foams have been satisfactorily used and equipment for their production is available.

Ease of Maintenance:

There should be no special maintenance problems for any of the sulfur categories.

Environmental Compatibility:

Possibilities of air pollution and personal harm or discomfort to humans exist with any sulfur usage.

Availability and Cost:

Availability is good. With no prototypes in existance, cost figures are not available. Costs of sulfur and its shipping are increasing at a substantial rate each year.

15.5.2 Phosphate Cements.

Ease of Construction:

Available as either cold setting or hot setting materials. Rapid setting times (5 to 10 min.) causes handling problems. When formulated with water, heat must be applied to remove the water. Not compatible with fresh concrete but suitable for use with hardened concrete.

Ease of Maintenance:

No unusual maintenance problems are expected.

Environmental Compatibility:

Ammonia gas is produced during setting of the cement but amounts are small and not considered serious for small amounts of phosphate cements or work out-of-doors.

Availability and Cost:

Limited availability and very expensive compared to conventional portland cement.

15.6 PORTLAND CEMENT CONCRETE

Concrete is a composite material which consists essentially of a binding medium within which are embedded particles or fragments of aggregate. In portland cement concrete the binder is a mixture of portland cement and water. The following is a summary of some of the characteristics of this material.

Compressive Strength:

Usual test employs a cylindrical specimen of height equal to twice the diameter, moist cured at 70 ±5F for 28 days, and then subjected to slow loading (2 to 3 minutes) until failure. Typical strength values of 2000 to 6000 psi obtained in this manner. Strengths as high as 10,000 psi are possible. Strength affected by many factors which include the cement, water-cement ratio, mixture proportions, aggregate size, curing age, rate of loading, and others. In practice, the largest single factor affecting strength is the water-cement ratio. Increasing water-cement ratios decrease strength. Various relationships between water-cement ratio and strength have been developed. More exact relationships between water-cement ratio and gel-space ratio also exist. Strength at any water-cement also depends on the degree of hydration of cement and its chemical and physical properties, the temperature at which hydration takes place, the air content of the concrete, and other factors. At a given water-cement ratio, increasing aggregate size reduces strength. Strength gain occurs with time with lower water-cement ratio concrete gaining faster than high water-cement ratio concrete. Strength gain related linearly to log of age from 3 days to 2 months. Curing also affects rate of strength gain with moist curing best. Load and strain rate sensitive.

Flexural Strength:

Commonly measured in pavement work. Expressed in terms of "modulus of rupture" calculated from a flexure formula. Typical specimen is 4 in. by 4 in. by 16 in. long supported a 12-in. span and loaded either at third points or at the center. Modulus of rupture ranges from 11 to 23 percent of the compressive strength being about 15 percent when compressive strengths are 3500 to 4500 psi. Flexural strengths are 60 to 100 percent higher than tensile strengths. Sensitive to test conditions and moisture content of concrete when tested. Crushed stone gives better results than smooth gravel. In general, affected by same factors which affect compressive strength.

Tensile Strength:

Seldom measured for routine work. Determined either directly or indirectly (splitting tensile). No direct proportionality between compressive and tensile strength with their ratio being primarily a function of the level of concrete strength. Ranges from 7 to 11 percent of compressive strength averaging 10 percent in compressive strength ranges of 3000 to 4000 psi. Decreases with increasing compressive strength. Sensitive to moisture content of concrete when tested.

Dimensional Changes:

Volume changes occur by plastic shrinkage (unhardened concrete), autogenous shrinkage or swelling, drying shrinkage, carbonation shrinkage, creep, and thermal expansion and contraction. Most changes are directly related to moisture conditions and availability. Other factors affecting changes are composition and fineness of the cement, cement and water content, type and gradation of aggregate, admixtures, temperature, and size and shape of specimens. Drying shrinkage values to 1200 millionths reported with 300 to 600 millionths more common. Sixty-six to 85 percent of 20-year shrinkage occurs in 1 year. Biggest influence is exerted by aggregate which restrains shrinkage. Carbonation shrinkage occurs after years of service. Creep normally not a consideration in pavements. Linear coefficient of thermal expansion varies from 3 to 8 millionths per degree F. Dependent on mixture composition and moisture content when measured.

Durability:

Affected by freezing and thawing, temperature variations, wetting and drying, reactive aggregates, aggressive chemicals, aggressive waters, and mechanical wear and abrasion. Resistance of concrete to disintegration by frost action and by the application of salts for ice and snow removal may be greatly increased by air entrainment of the concrete. Some aggregates may produce unsound concrete. These include readily cleavable, structurally weak, very absorptive, swelling and alkali-reactive aggregates. Sulfates of sodium, magnesium, and calcium present in soils and waters can react with hydrated lime and hydrated calcium aluminate in some concretes to produce disruptive expansions. Also adversely affected by most acids and chlorides, by ammonia nitrate, by a few coal-tar distillates, and most vegetable oils (surface destruction).

Abrasion Resistance:

Difficult to assess as the damaging action varies depending on the exact cause of wear. No one test is satisfactory. In general, abrasion resistance increases with compressive strength. Mixture composition is also relevant. Lowering the water-cement ratio through improvement of the aggregate grading improves resistance. Resistance to wear is also increased with the use of harder and tougher aggregates and reductions in the sand content. Good curing promotes improved resistance.

Toughness:

Generally little post-cracking behavior occurs hence toughness is principally influenced by the maximum strength obtained. Dependent on type of loading. Constant strain tests produce greater toughness of a given concrete than constant load tests on the same concrete. Toughness also influenced by the moisture at the time of test, the amount and modulus of the coarse aggregate, the shape of the coarse aggregate particles, and the surface characteristics of these particles. Crushed, angular aggregates and aggregates whose modulus of elasticity is high, produce improved toughness.

Fatigue Strength and Dynamic Conditions:

Concrete does not appear to have a fatigue limit. Fatigue strength is usually referred to at a given number of cycles. At a given number of cycles, fatigue failure occurs at the same fraction of ultimate strength and is thus independent of the magnitude of this strength and the age of the concrete. Believed to be the result of deterioration of bond between the cement paste and aggregate. Fatigue strength has been reported as 55 percent at 10 million cycles in flexure and 60 to 65 percent at 10 million cycles in compression. Concrete is both load and strain rate sensitive producing high strengths with corresponding increases in strain as rates increase.

Permeability:

Permeability of cement paste has the greatest influence on the permeability of concrete. It is a function of porosity and depends on the size, distribution, and continuity of the pores. Decreases with age.

Short-term permeability affected by individual cements and their influence on rate of hydration. Minor influence by aggregate. Reported values of 10^{-9} to 10^{-12} cm/sec.

Ease of Construction:

Widely used. Equipment and techniques for construction well developed. Most problems develop from ignoring proper techniques or using inferior materials.

Ease of Maintenance:

When properly designed and constructed and used only for the original design loadings, should remain relatively maintenance free. Depending on severity of the problem, maintenance can be fairly easy and routine (potholes and patches) or difficult and expensive (removal or overlays). Technology exists to accomplish both, however.

Availability and Cost:

Availability is not a problem with sources available for production in most areas of the United States. Portable production plants are also available for on-site production on any job. Costs for plant and labor will vary from job to job and location to location. Materials cost will also vary but can be expected to be from \$17 to \$25 per cubic yard of concrete.

15.7 EXPANSIVE CEMENT CONCRETE

Expansive cements are cements which, when mixed with water, form a paste that, after hardening, tends to increase in volume to a significantly greater degree than portland cement paste. They can be used to compensate for volume decrease due to shrinkage (shrinkage compensating cements) or to induce high tensile stresses in reinforcement (self-stressing cements). Shrinkage compensating cements are classified as Types M, S, or K, depending on composition. Self-stressing cements can also have the same classifications but with much higher expansive potentials or they can be produced by blending an expansive component (calcium sulfoaluminate (CSA)) with portland cements. Concrete made with any of the expansive cements or with additions of CSA is called expansive cement concrete.

Expansive cements, when properly used, induce a chemical compressive prestress in the concrete. For a crack to form, both the tensile strength of the concrete and the compressive prestress must be overcome. Self-stressing cements actually prestress reinforced concrete elements to levels comparable to those achieved with mechanical prestressing. The following is a summary of the characteristics of these materials.

Expansion:

Expansion is related to the expansive potential of the mixture and the amount and direction of restraint. Cement producer controls expansive potential of cement by adjusting chemical composition and fineness. User controls expansive potential of concrete by adjusting cement content, type of aggregate and admixture used, mixing time, type of curing, temperature during mixing and curing or both, and the degree of restraint provided by steel reinforcement.

Expansion Stresses:

Compressive stresses induced in concrete by shrinkage-compensating cements are from 15 to 100 psi while self-stressing cements may produce stresses from 100 to 1000 psi. Magnitude is related to the amount of expansion. Loss of prestress force due to shrinkage and creep are approximately equal to or less than those of mechanically prestressed concrete.

Compressive Strength:

Shrinkage-compensating concretes develop strengths equivalent in rate and magnitude to Type I or II portland cement concretes. Strength of self-stressing concretes is inversely related to the expansion of the cement which in turn is related to the amount of restraint.

Flexural Strength:

Similar to conventional concrete as described above for compressive strength.

Tensile Strength:

Similar to conventional concrete as described above for compressive strength.

Dimensional Changes:

Expansive behavior noted above. Drying shrinkage is not a function of expansion and depends on the usual parameters that affect that characteristic of conventional concrete. It is similar to shrinkage of Type I portland cement concrete. Coefficients of creep and thermal expansion within the same range of portland cement concretes of comparable quality.

Durability:

Freeze and thaw resistance affected by the same factors which affect portland cement concretes. Expansive cements made with Type II or Type V portland cement clinker and adequately sulfated, produce concretes having sulfate resistance equal to or greater than portland cement made of the same type. When made with Type I or III portland cement, they may be undersulfated and susceptible to possible deterioration from sulfate attack. This problem also exists for high additions of CSA.

Abrasion Resistance:

Comparable to portland cement concrete made with similar proportions and ingredients.

Toughness:

No change when used in unreinforced concrete. In reinforced elements where substantial prestress forces are developed, toughness of the element will be improved.

Fatigue Strength and Dynamic Conditions:

Dynamic response similar to comparable portland cement concretes. Data on fatigue life not available.

Permeability:

Less than that of portland cement concrete.

Ease of Construction:

No special problems. Effective curing is essential, however.

Ease of Maintenance:

No special problems for shrinkage compensating concrete pavements. Pavements should be more maintenance free than conventional pavements because of reduced cracking and icnreased joint spacings. For pavements with high levels of prestress, damage may cause loss of prestress, and with greater joint spacings or thinner sections or both, may precipitate further damage. In these instances more elaborate repair schemes may be necessary.

Environmental Compatibility:

No different than for portland cement concretes.

Availability and Cost:

No problem in availability except in a few areas of the United States where there are no local producing mills or distribution points. Can be shipped in to these areas, however. Basic unit cost of concrete expected to be higher than portland cement concrete, but costs will probably be offset by savings in reduced pavement sections or in construction costs by having substantially increased joint spacings.

15.8 FIBER REINFORCED CONCRETE

Fiber-reinforced concrete is defined as concrete made of hydraulic cements containing fine or fine and coarse aggregate and discontinuous, discrete fibers. Discrete fiber types include steel, glass, polymeric, asbestos, carbon, mineral wool, and vegetable fibers. Steel and glass fibers are presently best suited for pavement applications. Matrices include portland cement, high alumina cement, gypsum, and special cements such as expansive cements. Performance of fibers in concrete is related to fiber geometry, volume fraction of fibers in the mixture, fiber orientation, and bond development between fibers and the matrix. The following is a summary of the characteristics of these materials that are most suited for pavements.

Compressive Strength:

Additions of steel fibers to mortar generally reduce compressive strength from that of no-fiber mixtures. Ductility is improved, however. Strength of concrete generally increases with steel fiber volume (to 3 percent volume fraction) with increases of as much as 100 percent being reported. Increases of 10 to 25 percent are more realistic, however. The same behavior occurs for glass fiber reinforced concrete with optimum increases occurring between 0.5 and 1.0 percent volume fraction of fibers. Polymeric fiber additions reduce strength of concrete.

Flexural Strength:

Substantially improved (1.5 to 3 times) with additions of steel fibers. Maximum improvements occur in mortar and are reduced with aggregate additions. Affected by fiber variables such as geometry, volume fraction, and fiber surface condition. Limited data for glass fiber reinforced concrete but reported information indicates even greater improvements than for steel fiber reinforced concrete. Also affected by fiber geometry, volume fraction, and surface condition. Limited data exist for polymeric fibers but increases of 2 to 3 times have been reported for polypropylene fibers. Dependent on volume fraction of fibers and fiber length.

Tensile Strength:

Limited data for steel fiber reinforced concrete. Only small increases in strength observed. Similar findings for glass and polymeric fibers in concrete.

Dimensional Changes:

Limited data indicate that the contribution of all types of fibers in reducing drying shrinkage is small and may be negated when mixtures are adjusted to higher water and cement contents to accomodate higher fiber loadings. Fiber additions tend to reduce tensile creep but do not influence compressive creep.

Durability:

Related to the concrete constitutents and the interactions between them. Additions of fibers will not improve freeze-thaw resistance over what the concrete had prior to the fiber additions. Exposed steel fibers will rust. Steel fibers in uncracked or hairline cracked concrete perform satisfactorily. Glass fibers including alkali resistant glass fibers, deteriorate in a cementitious matrix. Long-term performance however has not yet been determined. Organic fibers such as polypropylene, nylon, and polyethylene are generally resistant to acids, alkalies, and water.

Abrasion Resistance:

Limited data, but indications are that steel fiber reinforced concrete is superior to portland cement concrete.

Toughness:

Limited quantitative data, but qualitatively the toughness is significantly improved (2 to 30 times) over portland cement concrete. Same parameters that influence ultimate strength also influence toughness, including fiber geometry, content, and orientation.

Fatigue Strength and Dynamic Conditions:

Flexural fatigue strength of steel fiber reinforced concrete increases with fiber volume. No change in compressive fatigue strength due to steel fiber additions. No data available on fatigue life of glass or polymeric reinforced concrete. Fiber reinforced concretes are load and strain rate sensitive.

Impact Resistance:

Substantial improvements for most kinds of fibers. Fibers prevent total disintegration and shattering of concrete normally associated with shock loads. Improvements of 3 to 4 times reported for steel fiber reinforced concrete. No data available for glass fiber reinforced concrete. Polymeric fibers in concrete improve impact resistance 20 to 30 times. Improvements are related to interfacial bond of the fiber with the poor bonds producing the best results.

Permeability:

Fiber additions by themselves should not significantly affect permeability. Adjustments in the proportioning of the ingredients of the concrete to accommodate the fiber additions, such as changing the water-cement ratio and cement content, may cause the permeability to change however.

Ease of Construction:

More difficult to handle, batch, mix, and place than conventional concrete. Additional labor and equipment needed. Production and placement rates slower than conventional concrete.

Ease of Maintenance:

More effort will have to be expended to remove a fibrous concrete section than a plain concrete or reinforced concrete section.

Environmental Compatibility:

No problems foreseen.

Availability and Cost:

Availability of steel and glass fibers is good. Limited availability on other types. Substantial increases in materials costs with smaller increases in labor costs. These may be offset by cost reductions due to thinner sections and increases in joint spacings.

15.9 POLYMERS IN CONCRETE

The concretes which utilize polymers to form composite materials have been generally categorized as polymer-impregnated concretes (PIC), polymer-portland cement concrete (PPCC), and polymer concrete (PC). The following definitions describe these materials:

Polymer-Impregnated Concrete:

Polymer-impregnated concrete (PIC) is a precast portland cement concrete impregnated with a monomer which is subsequently polymerized *in situ*.

Polymer-Portland Cement Concrete:

Polymer-portland cement concrete (PPCC) is a premixed material in which either a monomer or polymer is added to a fresh concrete mixture in a liquid, powdery, or dispersed phase, and subsequently cured and if needed, polymerized in place.

Polymer Concrete:

Polymer concrete (PC) is a composite material formed by polymerizing a monomer and aggregate mixture. The polymerized monomer acts as the binder for the aggregate.

15.9.1 Polymer Impregnated Concrete (PIC). PIC can be produced using any hardened concrete which first must be dried to remove water and provide voids for monomers to infiltrate. Various techniques are available to introduce the monomers into the concrete. Polymerization is accomplished by either radiation, thermal-catalytic, or promoter-catalyst techniques. The following is a summary of some of the characteristics of PIC.

Compressive Strength:

Increases of as much as four times over control strengths. Final strength dependent on extent of the impregnation and filling of pores, the type of polymer and its ability to carry stress, and the degree of conversion of monomer to polymer during polymerization. PIC strengths and largely independent of the quality of the initial concrete provided full impregnation is achieved.

Flexural Strength:

Same behavior as for compressive strength.

Tensile Strength:

Same behavior as for compressive strength.

Modulus of Elasticity:

Fully impregnated concretes have shown increases in both elastic and flexural moduli of as much as 40 percent. Partially impregnated concretes show little change.

Dimensional Changes:

Thermal coefficient of expansion of PIC has been observed to be as much as 25 percent greater than conventional concrete but in general it is not significantly greater. Large shrinkages occur during polymerization. Compressive and tensile creep is reduced for PIC at normal and elevated temperatures.

Durability:

Improved over conventional concrete. Attributed to filling of the concrete pore system. Freeze-thaw and chemical resistance improved. Gradual reduction in strength occurs with increasing temperatures.

Abrasion Resistance:

Modest improvement.

Toughness:

No change or slight decrease.

Fatigue Strength and Dynamic Conditions:

Data not available.

Permeability:

Resistance to liquid intrusion is greatly improved.

Ease of Construction:

Requires complete construction of the original concrete, with subsequent drying, impregnation, and polymerization. Time consuming and expensive process. Special equipment and techniques required.

Ease of Maintenance:

Special equipment and techniques are needed to use PIC to repair a PIC pavement. Time-consuming operation.

Environmental Compatibility:

Limited information available. Some safety problems associated with monomer use and elevated temperature or radiation polymerization treatments. Small amounts of toxic fumes can be produced.

Availability and Cost:

Availability affected by fluctuations in the petro-chemical industry. Total cost of PIC in-place expected to be substantially more than a conventional portland cement concrete pavement or bridge deck.

15.9.2 Polymer-Portland Cement Concrete (PPCC). Polymer-portland cement concrete (PPCC) has been prepared with both pre-mix polymerized and post-mix pooymerized materials. The pre-mix polymerized materials include latexes and polymer solutions or dispersions. The post-mix polymerized PPCC has been made with a number of resins and monomers. The fabrication of PPCC is very similar to that of conventional portland cement concrete. Organic materials in either a powdery or dispersed form are added to the mixture during mixing. Not many organic polymers are compatible with freshly mixed concrete, however. The following is a summary of the characteristics of PPCC.

Compressive Strength:

Affected by state of moisture surrounding concrete. Slight strength improvements exist for air-dry curing. Wet curing produces slightly lower strengths.

Flexural Strength:

Moderate improvements in both strength and increased strain at failure. Also affected by moisture with air-dry curing needed for best results.

Tensile Strength:

Same as for flexural strength.

Dimensional Changes:

Drying shrinkage magnitude dependent on polymer type and loading. Wide variations in reported results being both more or less than conventional concrete. Limited creep data indicate greater creep than conventional concrete. Thermal coefficients are similar to portland cement concrete.

Durability:

Substantial improvement over portland cement concrete. Attributed to lower water contents during fabrication and reduced porosity. Good freeze and thaw resistance. Chemical resistance depends on the nature and amount of polymer and the chemical in question. Rapid loss of mechanical properties at elevated temperatures.

Abrasion Resistance:

Substantial improvements over portland cement concrete.

Toughness:

Significant increases in ductility and toughness.

Fatigue Strength and Dynamic Conditions:

Data not available.

Permeability:

Mixed findings reported as being improved and not improved. Result depends on whether contact with moisture puts polymers back into dispersed phase.

Ease of Construction:

Can be made and placed as simply as conventional concrete. Cleanup of equipment more difficult due to excellent adhesion of polymers. Special equipment required for post-mix polymerization.

Ease of Maintenance:

No special problems.

Environmental Compatibility;

Limited information available. Not expected to create problems, however.

Availability and Cost:

Availability generally good but affected by fluctuations in the petro-chemical industry. Costs of PPCC increased over portland cement concrete by 20 to 25 percent.

15.9.3 Polymer Concrete. Polymer concrete is a composite material consisting of a polymer matrix and particular fillers, prepared by the integral mixing of a polymerizable material (such as monomer or resin) and aggregate. Polymerization is usually obtained through a catalyst-promoter system without the introduction of radiation or thermal energy. Various polyesters, epoxies, furans, and PMMA have been used as the matrix of PC because of the reasonable compromise between relative ease of polymerization and desirable properties. Most of the work has been done with polyester-styrene resin systems. Most monomer and resin systems for PC are polymerized at room temperatures using promoter-catalysts although heat and radiation can be used. The properties of PC are largely dependent upon the properties of the polymer binder and the amount of polymer in PC, modified somewhat by the effects of aggregates and filler materials. The following is a summary of some of the characteristics of PC:

Compressive Strength:

Strengths are considerably greater (2 to 5 times) than those of portland cement concrete. Dependent on filler content, increasing with filler content until compaction becomes difficult. Temperature sensitive, decreasing with increasing temperatures.

Flexural Strength:

Significantly improved (4 to 10 times) over those of portland cement concrete. Affected in the same manner as described for compressive strength.

Tensile Strength:

Improved to the same extent and affected by the same things as compressive strength.

Dimensional Changes:

Considerable shrinkage during hardening. Controlled by filler additions. Thermal coefficients substantially higher than portland cement concrete, but decreases with aggregate filler additions. Large creep strains which increase rapidly (several hundred percent) with temperature increases.

Durability:

Excellent chemical resistance. Data on freeze and thaw resistance not available. Loses structural integrity and strength at elevated temperatures.

Abrasion Resistance:

Data not available, but experience indicates good performance.

Toughness:

Wide variation in PC composites. Dependent on polymer constitutents and temperature.

Fatigue Strength and Dynamic Conditions:

Data on fatigue resistance not available. Behavior affected significantly by rate of loading with rapid rates producing a nearly linear behavior and slow rates rates, a viscoelastic behavior.

Permeability:

Essentially zero when properly formulated. Significantly better than portland cement concrete.

Ease of Construction:

Batching, mixing, and placing techniques for producing PC are largely based on adaptations of existing equipment and methods for producing portland cement concrete. Equipment cleanup is difficult. Special mold or form release agents needed. If thermal-catalytic or radiation techniques are used for polymerization, special equipment is needed.

Ease of Maintenance:

No special problems foreseen.

Environmental Compatibility:

Limited information available. Many of chemicals used are irritants and toxic. Heat and radiation sources for polymerization may also present some problems.

Availability and Cost:

General availability is good but subject to fluctuations in the petro-chemical industry. Very expensive. Costs vary depending on type of monomer or resin system used and the amount of polymer used in the composite.

15.10 SEALANT MATERIALS

Sealants were grouped into four categories: thermosetting polymers (liquids), asphaltic materials, other resins (liquids), and sheet materials. More specifically, the various categories included: thermosetting polymers - epoxy resins, polyesters and polyurethanes; asphaltic materials - mastic asphalt, asphaltic concrete, coal tar, and rubberized asphalt; other resins (liquid) - linseed oil, silicones, chlorinated rubber, latex, and waxes; and sheet membranes - rubber sheets, polymer-impregnated fabric sheets, bituminous coated membranes, and modified coal tar reinforced with fabric. The purpose of sealants is to prevent the penetration of water or corrosive solutions into pavements and improve frost resistance, increase strength of the top surface, and reduce wear. These sealants cannot be rated in the same manner as the previous materials because of their form when used. They are not the structural portion of the pavement, hence the usual structural properties of the materials are not measured. The following is a summary of the characteristics of these materials as they might perform as a sealant.

Epoxy Resins:

Widely evaluated as a sealant. Numerous types of epoxies tried with flexible epoxy systems being best suited for pavement work. Tensile strengths and elongations range from 435 to 3655 psi and 10 to 104 percent, respectively. Compressive strength ranges from 1280 to 10,230 psi. Hardness (Shore D) range is 28 to 77. Water absorption varies from 0.17 to 2.1 percent. In general, not impermeable. May lose flexibility with time. Does not resist cracking as well as other types of membranes.

Polyesters:

Limited evaluations as protective coatings for pavements. Wide range of physical properties. Reported tensile strengths and elongations of 900 to 3960 psi and 23 to 85 percent, respectively. Reported compressive strengths range from 3240 to 5100 psi while water absorptions were 0.75 to 1.2 percent. Do not adhere well to concrete.

Polyurethanes:

Limited evaluations. Reported tensile strengths and elongations range from 2200 to 5000 psi and 350 to 800 percent respectively. Bond to concrete is poor and requires a primer coat. Evidence of blisters and pinholes during application.

Mastic Asphalt:

More imprevious than asphaltic concrete but still permeable. Some problems with blowing and blistering during application. Limited information available.

Asphaltic Concrete:

See Section 15.1. Generally not satisfactory as sealant.

Coal Tar:

Limited information. Poor field performance. Formed pinholes and blisters during placing and did not seal the surface. Inadequate flexibility. Delaminations occurred when exposed to water and freeze and thaw cycles.

Rubberized Asphalt:

Similar performance to coal tar.

Linseed Oil:

Most widely used protective coating for concrete. Varying results depending on climate. Proper applications will protect concrete through approximately 50 freeze-thaw cycles (1 to 2 years exposure). Reduces scaling of nonair-entrained concrete. Penetration depth of only one one-hundredth of an inch. Temporary sealant.

Silicones:

Limited information. No significant effect on the durability of concrete. Does not prevent scaling of nonair-entrained concrete. Does not prevent against deicer chemicals.

Chlorinated Rubber:

Limited information. Same behavior as silicones.

Latex Modified Concrete:

See Section 15.9.2. Good field performance.

Internally Sealed Concrete (Waxes):

Limited information. Effectively seals concrete. Reduces scaling. Special procedures involved to cause the sealing once the concrete is in place. Can be sealed internally (silicones and waxes) or externally (polymers or sulfur).

Sheet Membranes:

Flexibility and resistance to cracking are good. Relatively impermeable. Special treatment needed to ensure adequate bond to pavement.

15.11 CERAMIC MATERIALS

In broad terms, a ceramic material is any product made from a nonmetallic mineral by firing at high temperatures. Considered in this report were calcined bauxite, pozzolans, portland-blast furnace slag cement or granulated slag, expanded clay, shale and slate, cement clinker, and refractory shapes. Only calcined bauxite and pozzolans were considered as being candidate materials for a zero maintenance pavement. Emphasis was placed on calcined bauxite because of its much broader range of applications. It consists primarily of corundum (alpha alumina) which is a hard material of high elastic modulus and is available in aggregate form. The following is a summary of some of the characteristics of the material.

Compressive Strength:

Strength of polycrystalline alumina ranges from 260,000 to over 400,000 psi and is a function of porosity. Concrete made with calcined bauxite aggregate produces very good strengths.

Flexural Strength:

Bending strength of polycrystalline alumina ranges from 20,000 to 36,000 psi, diminishing with increasing porosity. Concrete made with calcined bauxite aggregate is expected to produce flexural strengths better than those of concretes made with conventional aggregates.

Tensile Strength:

Tensile strength of polycrystalline alumina, 95 percent dense, is approximately 22,000 psi at room temperature. Concrete made with calcined bauxite aggregate is not expected to have its tensile strength significantly improved over concrete made with conventional aggregates.

Dimensional Changes:

Not expected to be significantly different in the unhardened state than concrete made with ordinary aggregates. Little shrinkage or swelling or creep is expected as is true in concretes made with aggregates of high elastic modulus.

Durability:

The soundness of calcined bauxite is good and should not create problems during freezing and thawing. Alumina is described as "very slightly soluble" in acids and alkalies. Very fire resistant.

Abrasion Resistance:

Mohs scale hardness of alumina is 9 and Knoop indent hardness is 2000. Both indicate excellent resistance to abrasion. Calcined bauxites have polished stone values of 75 and aggregate abrasion values of 3.0, both which also indicate high aggregate toughness and abrasion resistance. Field performance as toppings for pavements substantiates this.

Toughness:

Information not available.

Fatigue Strength and Dynamic Conditions:

Informaiton not available.

Permeability:

Should not be significantly different than conventional concrete made with a nonporous aggregate.

Ease of Construction:

No special problems anticipated.

Ease of Maintenance:

No special problems anticipated.

Availability and Cost:

Most sources outside the United States. If used as the sand fraction of concrete, can be expected to increase the materials cost for concrete by $70 or $100 per cubic yard of concrete.

15.12 PRESTRESSED CONCRETE PAVEMENTS

Prestressing concrete pavements introduces internal stresses of such magnitude and distribution in the concrete that tensile stresses resulting from service loads are counteracted to a desired degree. Additional advantages include reducing the amount of transverse joints, absence of cracks in the road surface, resultant reduction of moisture in the road foundation, reduction of warping and curling stresses, and reduction in slab thickness. Compared to conventional portland cement concrete pavements the following characteristics of prestressed concrete pavements should be noted.

Slab Lengths:

Lenghts greater than 300 ft but not more than 700 ft are recommended.

Slab Thickness:

Pavement thickness can be reduced as much as 50 percent but a minimum thickness of 5 in. appears to be required.

Subgrade Interaction:

Because of thinner sections, prestressed concrete pavements are more flexible thus distributing load more effectively over the subgrade and resulting in reduced pavement stresses under wheel loads. The exact relationship between the prestressed pavement and the subgrade interaction is still unknown, however.

Subgrade Friction:

This is the major adverse factor in the prestress system. Various techniques to reduce this friction have been used.

Joints:

Joints between prestressed slabs constitute the area of most concern to prestress pavement designers. Joints cannot be too rigid. A durable, trouble free joint design has not yet been proven.

Ease of Construction:

The technology to construct prestressed concrete pavements is available although standard design procedures are still being developed. Will require more effort to construct than a conventional concrete pavement.

Ease of Maintenance:

Repair to small areas can be readily made but major repairs will be difficult. Damage to prestress pavements and especially to the reinforcing (prestressing) elements will be more detrimental than to a conventionally reinforced pavement and will call for more elaborate repair procedures.

Availability and Cost:

Availability is no problem. Indications are that prestressed concrete pavements can be constructed on a competitive basis with conventional concrete pavements and will be more economical to maintain.

15.13 VACUUM-PROCESSED CONCRETE

Vacuum-processed concrete is portland cement concrete, mixed and placed in the usual manner, but immediately subjected to a vacuum applied to one or more surfaces through vacuum chambers in contact with the unhardened concrete thus resulting in removal of a significant portion of the mix water. This removal of the excess "water of convenience" from the unhardened mixture results in a reduction of the final water-cement ratio and a general improvement of most physical properties of the concrete. Two parameters affect the removal and these are the initial water-cement ratio of the concrete and the time of processing or vacuuming. The following is a summary of the characteristics of vacuum-processed concrete.

Compressive Strength:

Increases of 40 percent may normally be expected when processing conventional concrete. If vibration can be incorporated in the vacuum processing, additional water can be removed and higher strength achieved. Removal of more than 20 percent of the mix water may be unwarranted, however.

Flexural Strength:

Limited data available. Increases of 25 percent have been reported.

Tensile Strength:

Limited data available. Increases of 25 percent have been reported.

Dimensional Changes:

Reductions of 30 to 50 percent have been reported. Indications are thatthe creep will be proportionally reduced with water reduction by vacuum processing.

Durability:

Improved frost resistance over conventional concrete. Reduced deicer penetration. Improved resistance to deicer scaling.

Abrasion Resistance:

Improvements up to 300 percent have been reported. Surfaces treated by vacuum processing have been observed to wear 16 to 39 percent less than traditionally laid floors.

Toughness:

Information not available.

Fatigue Strength and Dynamic Conditions:

Information not available.

Permeability:

Less than that obtained with conventional concrete.

Ease of Construction:

Special equipment and techniques required although the system is not complicated. Can be integrated into a conventional paving train. Power troweling is required.

Ease of Maintenance:

No special problems.

Availability and Cost:

Limited availability in United States, but equipment is available through United States outlets. Cost figures are generally not available but the in-place cost may be more than conventional concrete costs. These may be more than offset by reduced maintenance costs.

15.14 RATING OF MATERIALS

To rate all the materials and materials systems described in this report on a common basis is extremely difficult and, for the most part, not feasible. This is because the various materials and materials systems perform in different manners and are used in different designs which take advantage of the best features of each material or materials system. For example, in rating of compressive strength, asphalt would always come out second best to concrete although its compressive strength is apparently adequate in most instances. Sealants and prestressed concretes will probably never have any meaningful compressive strength information. The same problems exist for most other characteristics of each material and materials system.

In order to narrow the field of comparisons, the questions are posed as to what can be done to a flexible pavement system (asphalt based) or rigid pavement system (portland cement concrete and other rigid binder systems) to make them less susceptible to deterioration and hence more like a pavement requiring reduced or no maintenance. The rating system used to highlight the good and bad features of each candidate material involves the following simple notation:

P = Poor
NC = No Change
NA = Not Available
G = Good
VG = Very Good

In relative terms, poor (P) means that this characteristic is not as good as the basic material, that is, either asphalt concrete or portland cement concrete as the case may be. Strengths or other properties may be lower or not as effective, costs may be higher, it may not be available, or it may take more effort to construct or maintain it. No change (NC) indicates no significant change in the characteristic of the material or materials systems. Not available (NA) means that sufficient information was not available to make a reasonable comparison. Good (G) indicates a reasonable improvement in mechanical properties, reduced costs, or easier construction or maintenance. Very good (VG) indicates an exceptional improvement of the characteristics in question.

The characteristics to be rated form the basic format for this report and include:

Temperature Dependency
Compressive Strength
Marshall Stability
Flexural Strength
Tensile Strength

Modulus of Elasticity
Dimensional Changes
Durability
Impact Resistance
Abrasion Resistance
Toughness
Fatigue Strength
Dynamic Response
Permeability
Ease of Maintenance
Ease of Construction
Environmental Compatibility
Availability
Cost

Each material or materials systems may not have had each characteristic rated either because information was not available or that characteristic was not directly germane to the performance of the material or materials system in a pavement.

15.14.1 Asphaltic Based Materials. In addition to asphalt concrete, the asphaltic based materials include gussasphalt, asbestos asphalts, and sulfur modified asphalts. The comparison or rating of the characteristics of these materials compared to those of the asphalt concrete is shown in Table 15.1. In many instances, there was insufficient information available to make comparisions of all characteristics.

All of the asphaltic based materials are temperature dependent, their performance being significantly affected by the temperature at which they are evaluated.

Gussasphalt has proven to have better compressive and tensile strengths depending on temperature. Its durability, abrasion resistance, and permeability are improved. There is no significant difference in its ease of construction, ease of maintenance, and environmental compatibility. It is not as readily available as asphaltic concrete and it costs more. Information on marshall stability, flexural strength, dimensional changes, toughness, fatigue strength, and dynamic response was not available.

Asbestos asphalts show an improvement in strength, durability, toughness, and fatigue resistance. The presence of the asbestos has no significant effect on marshall stability, permeability, construction, and maintenance. The handling of the asbestos may create some environmental problems not associated with asphalt concrete. It is not readily available and costs more. Information on dimensional changes, abrasion resistance, and dynamic response was not available.

Sulfur modified asphalts have shown improvements in marshall stability and permeability and an exceptional improvement in tensile strength. The fatigue strength has been reported as being both improved and adversely affected by the presence of the sulfur. There is no change in the flexural strength, in ease of maintenance or availability. It requires more effort during construction, fume emissions may create an environmental problem, and it costs more. Information was not available on compressive strength, dimensional changes, durability, abrasion resistance, toughness, and dynamic response.

15.14.2 Special Concretes and Rigid Binder Systems. The special concretes include phosphate cement concrete, expansive cement concrete, fiber reinforced concrete, polymer impregnated concrete, polymer-portland cement concrete, sulfur-infiltrated concrete, prestressed concrete, vacuum processed concrete, calcined bauxite aggregate concrete, and internally sealed concrete. Rigid binder systems include sulfur concrete and polymer concrete. The comparison or rating of the characteristics of these materials compared to those of portland cement concrete is shown in Table 15.2 In the case of prestressed concrete, the only characteristics which could be compared were the ease of maintenance and construction, environmental compatibility, availability, and cost as the majority of the conventional mechanical properties are usually never determined for concrete in the prestressed state.

When sulfur replaces cement as the binder in concrete, as is the case with sulfur concrete, there is no advantage gained in compressive and flexural strength, modulus of elasticity, ease of maintenance and construction, and availability. Dimensional stability, durability and environmental compatibility all worsen. Permeability is significantly reduced. Information on tensile strength, abrasion resistance, toughness, impact resistance, fatigue strength, dynamic response, and cost were not available.

Sulfur-infiltrated concrete showed significant improvements in compressive strength and permeability and modest improvements in modulus of elasticity and durability. No change occurred in the level of availability and the ease of maintenance. Both the ease of construction and the environmental compatibility worsened. All other characteristics did not have sufficient information, if any, to make a comparison.

The use of phosphate cements in lieu of portland cements offers no significant improvements or advantages in any characteristic. The ease of maintenance and construction, environmental compatibility, availability, and cost all worsened. Information on durability was not available.

Polymer impregnated concrete offers advantages similar to those of sulfur infiltrated concrete. All strengths were significantly improved with moderate improvements being noted in modulus of elasticity. Durability is improved along with significant reductions in the permeability. Dimensional stability varies from no change to a decrease in performance. No change occurred in abrasion resistance, toughness, and availability. The ease of maintenance and construction, the environmental compatibility, and cost worsened. Comparisons of impact resistance, fatigue strength, and dynamic response were not possible.

Polymer-portland cement concrete shows moderate improvements in flexural and tensile strengths, abrasion resistance, and toughness. Significant improvements occurred for durability. Compressive strength and dimensional changes varied from poor to no change, while permeability varied from poor to good depending on polymer type. No change was noted for ease of maintenance and construction, for environmental compatibility, and for availability. Costs worsened. Comparisons for modulus of elasticity, impact resistance, fatigue strength, and dynamic response were not possible.

The replacement of the portland cement binder with a polymer binder in polymer concrete produced good to very good improvements in strength. Moderate improvements in durability and significant reductions in permeability also occurred. Toughness varied from poor to good depending on polymer types. The dimensional changes, ease of construction, and cost all worsened. No change was noted for dynamic response and availability. Comparisons for modulus of elasticity, abrasion resistance, impact resistance, fatigue strength, and environmental compatibility could not be made.

The use of expansive cements in lieu of portland cements in concrete produced no significant changes in all characteristics except dimensional changes where slight improvements occurred, permeability where good reductions occurred, and durability where ratings of poor to no change occurred. Information on fatigue strength was not available.

The addition of fibers to concrete substantially improved the flexural strength, toughness, and impact resistance. Moderate improvements in abrasion resistance and fatigue strength also occurred. Compressive strength, tensile strength, and durability varied from no change to a good improvement. No change was noted for modulus of elasticity, dimensional changes, dynamic response, permeability, environmental compatibility, and availability. Ease of maintenance and construction and cost all worsened.

The use of calcined bauxite as the sand in concrete produced improvements in compressive and flexural strength and abrasion resistance. Availability and cost worsened. All other characteristics showed no change except modulus of elasticity, toughness, impact resistance, fatigue strength, and dynamic response where no comparisons could be made.

As noted previously, comparisons could not be made for most characteristics of prestressed concrete. The ease of maintenance and construction of these pavements is more difficult than portland cement pavements and is hence rated poor. No change in the environmental compatibility, availability, and cost occurred.

Vacuum-processed concrete shows improvements in strength, dimensional changes, and durability; with permeability being reduced. Substantial improvements occurred for abrasion resistance. No change was noted in ease of maintenance, environmental compatibility, and cost. Ease of construction and availability worsened. Comparisons for all other characteristics could not be made.

Information on internally sealed concrete was limited. Compressive strength was not changed nor was ease of maintenance, environmental compatibility, and availability. Ease of construction and cost worsened. Permeability and modulus of elasticity improved. No other comparisons could be made.

15.4.3 Other Materials and Materials Systems. The sealants and sulfur-surface applications did not lend themselves to the above rating system and were not compared to either the asphalt concrete or portland cement concrete.

15.15 CONCLUSIONS AND RECOMMENDATIONS

15.15.1 Gussasphalt. Gussasphalt, a thin, voidless bituminous surface course, has provided long service on many heavily trafficked highways in Europe. Pavements 8 to 10 years old with 87,000 to 124,000 vehicles per day are relatively free of reflective cracking and rutting deformation.

Gussasphalt shows promise as a paving material with a long service life potential with little or no maintenance requirements. It is generally considered to be stronger than a conventional asphalt concrete. Gussasphalt also has other advantages such as:

a. Impervious to water.

b. Monoplastic under normal temperatures.

c. Compaction of mixture not required.

d. High abrasion resistance.

The lack of experience in the design and construction and the need for special equipment to place a gussasphalt pavement in the United States would result in a high initial investment. Even without the high initial expense, the cost of placing gussasphalt would be approximately twice that of a conventional asphalt concrete, but the expected service life of the gussasphalt pavement should be longer with less maintenance requirements.

The addition of additives such as asbestos, fiberglass, or rubber indicates some improvement of the physical properties of a gussasphalt, but substantiating data on such additives were not available.

It is recommended that gussasphalt be given further consideration as a potential zero maintenance material.

15.15.2 Asbestos Asphalts. The use of asbestos fibers allows the asphalt content to be increased without losing many of the physical properties of the bituminous mixture. Properties such as compressive and tensile strength were improved by the addition of asbestos fibers, and the asbestos modified mixtures showed an increase resistance to densification (rutting). Because of the resistance to densification, initial compaction may be more difficult. If the required compaction cannot be obtained, then the asbestos modified mixture may have a high water permeability.

The addition of asbestos to a bituminous concrete should make the resulting mixture more durable and less likely to deteriorate under traffic, but the resulting pavement will not be maintenance free. The amount and degree of maintenance will generally be less than expected with a conventional asphaltic concrete pavement.

15.15.3 Sulfur Modified Asphalts. The addition of sulfur to an asphaltic mixture improves many of the physical properties, such as stability, flow, and permeability, but because of limited testing that has been conducted, there could be some disadvantages that have not been uncovered at this time. Additional extensive testing, under various test conditions, is dictated. Low and high temperature properties need further investigating, and field performance needs evaluating. Sulfur mixtures do not appear to be maintenance free and cannot be recommended as a zero maintenance material until additional testing has been completed.

15.15.4 Noncalcareous Inorganic Cements.

15.15.4.1 Sulfur. While numerous references to the use of sulfur exist, the data contained in them is not extensive and is insufficient to make reasonable judgments as to the suitability of sulfur as a zero maintenance material. Sulfur impregnation or infiltration may be a suitable technique for excluding moisture from the pavement and this is highly desirable. Foamed sulfur may be useful in preventing frost heave damage. Not enough is known about sulfur admixtures and sulfur concrete to judge their acceptability for pavement usage. It is recommended that additional testing and evaluation of sulfur be done before deciding on its zero maintenance potential. This testing should include the following:

a. Immersion of nondurable porous aggregates in molten sulfur to reduce their absorption. These coated aggregates should then be used in concrete mixtures. Specimens made from these mixtures should be tested in freezing-and-thawing and other tests to evaluate the effect of this coating on durability and other concrete properties.

b. Development of procedures to impregnate or infiltrate in-place concrete should be pursued so as to utilize the beneficial effects of sulfur void filling in hardened concrete.

c. Concurrent testing and evaluation of the effects of applying molten sulfur to concrete surfaces by brushing or spraying to improve concrete properties without the use of vacuum should be done.

15.15.4.2 Phosphate Cements. The fact that these systems require special aggregates (i.e., periclase, wollastonite) means that supplies and overall cost would probably limit usage to patching applications. In addition, it was pointed out that these systems are not compatible with fresh concrete because of the need for heat to remove water or for the need to avoid water for cold setting systems. It would therefore seem that their promise lies in use as patching materials for rapid and high strength repairs to existing conventional concrete structures or roads. They are not recommended for consideration as a candidate zero maintenance material.

15.15.5 Expansive Cement Concrete. Expansive cement concretes, when used properly, should help in reducing the maintenance problems associated with concrete pavements. Shrinkage-compensating cement concrete pavements should allow present joint spacings to at least double while remaining crack free from the effects of drying shrinkage. With the number of joints at least reduced in half, the problems associated with joints and their maintenance are also reduced in half. The crack-free pavement should also resist the ingress of aggressive solutions and thus the problems associated with them.

Self-stressing cement concrete pavements should improve even further on the reduction of joints. The advantages and disadvantages of mechanically prestressed concrete pavements are described in Chapter 13 and are equally applicable to chemically prestressed pavements.

The use of shrinkage-compensating cements in concrete should pose no major problems with regard to equipment and techniques used. How to use the material is already understood and the problem is just to transfer this knowledge to the user. Self-stressing cement use in pavement concrete still has a long way to go. The problems associated with its use are not unique. With an understanding of the chemically prestressing mechanism and the material characteristics of the cement and concrete, these problems can readily be overcome, however. The idea of the use of expansive components to produce tailor-made behaviors is excellent and has a lot of merit. Its implementation will be difficult on a broad basis, however. The major obstacle appears to be the liability for getting the proper expansion. Presently, concrete suppliers are somewhat insulated from the responsibility of achieving proper concrete behavior, because the cement company makes a product that, when used in proper amounts and given proper treatment by the user, should produce proper results. With an expansive component, the responsibility shifts to the supplier (as it does with all admixtures) to ensure that the proper amount is added to the mixture.

Expansive cement concrete, while reducing pavement maintenance problems, cannot be expected to result in a zero maintenance pavement when used by itself. It may be more useful when in combination with other materials or processes such as internally sealed concrete (Chapter 11) or polymer impregnation (Chapter 10). Additional work on combinations of this sort is recommended before eliminating this material from further consideration.

15.15.6 Fiber-Reinforced Concrete. Fibrous concrete appears to have considerable merit for use in overlays, pavements, and bridge decks. Of the many types of fibers available, only steel and perhaps glass would be directly suitable for use in concrete that would represent the wearing surface. Polymeric fibers could be effectively used in areas of impact loading such as curbs and gutters. Not enough is known about carbon fibers to comment on them suitably. Asbestos fibers will probably not be satisfactory.

Most of the fibers impart improvements to pavement performance beyond that of just increasing the flexural strength of the pavement. The improved toughness and crack arrest potential are ideally suited for pavement loadings. As a zero maintenance material it is significantly better than plain concrete, although most actual installations have experienced some cracking. For the most part these cracks have remained very tight and have not developed into working cracks. The use of fibrous concretes allows a considerably reduced frequency of joints which in turn will considerably reduce the maintenance problems associated with joints.

The use of steel fiber reinforced concrete is not without some problems. Production of the concrete involves a limited amount of special equipment, major proportioning changes, and special batching techniques. None of these problems are insurmountable, however, but must be handled on an individual job basis. Maintenance of fibrous concrete pavements or overlays, should it be necessary, will be considerably more difficult than plain or reinforced concrete pavements or overlays. Once a working crack develops, there is nothing to tie the two sections on each side of the crack together. Feather-edging of repairs is not possible. Edges of repairs must be at least one inch in depth so the concrete won't unravel. Fibrous concrete removal is difficult using conventional means because of its ability to absorb impact and hang together once cracked.

In general, the steel fiber reinforced concrete has performed best in new construction of full depth pavements. These pavements have been reduced in thickness from conventional designs to make the concrete cost in place more attractive. It is entirely possible that by going to even thicker sections of fiber reinforced concrete, the pavement would begin to approach a relatively maintenance free condition. Combinations of the thick pavement with techniques for sealing the voids of the concrete as described in Chapters 5, 10, and 11 may produce an excellent zero maintenance candidate material. It is recommended that fibrous concretes, and in particular, steel fiber reinforced concrete, be investigated further as a potential material for use in zero maintenance pavements.

15.15.7 Polymers in Concrete. The PIC appears to make a significant contribution to the concept of zero maintenance, although in its present form it is not a zero maintenance material. PPCC shows some improved durability characteristics but these are not significant enough to qualify it, it its present form, as a zero maintenance material. PC has considerable merit as a potential zero maintenance material, but not enough is known about its long-term performance to judge it at this time.

It is recommended that further consideration be given to both PIC and PC as candidate zero maintenance materials. Both are expensive and there are problems associated with their fabrication. Their mechanical properties are good, however. They could possibly be used in combination with other materials to result in an even better zero maintenance material.

15.15.8 Sealant Materials. Twenty-one manufacturers of coatings and sealants were contacted in regard to their recommendations and available materials for pavement sealants. Nine of the manufcaturers recommended four different polyurethane systems, three epoxy resin systems, a latex concrete, linseed oil emulsion, acrylic epoxy emulsion, and two sheet membranes. Some of these sealants have been evaluated by different organizations as pavement sealants. The four polyurethanes and acrylic epoxy emulsion had never been evaluated as pavement sealants but were used to seal parking decks and aircraft carrier decks. The manufacturers claimed that the performance of these materials had been satisfactory since the time of application.

Eight state highway departments were contacted in regard to their recommendations of pavement sealants. Most of the state highway departments are using linseed oil, and they claim that the oil treatment helps in reducing the amount of scaling and spalling of pavements. One of the states no longer uses linseed oil and claims that treatments with the oil lowers skid resistance. Two states are presently investigating impregnated concrete pavements with methyl-methacrylate and believe this material is promising as a sealant.

One state highway department claimed that proper air-entrained concrete was the best method and that sealants wouldn't be necessary. Other state highway departments contacted are evaluating epoxy-coated reinforcement bars, latex modified concrete, and sheet membranes.

A number of highway departments and manufacturers of sealants do not believe it would be feasible to seal the top surface of pavements (on grade) with an impervious sealant. An impervious sealant would prevent water vapor from passing through the pavement, therefore, the pavement would stay saturated with water absorbed from the subbase for long periods of time. To completely seal a pavement, the pavement would have to be enveloped with a sealant material or it could be internally sealed by the addition of polymers to the concrete mix before placement.

Laboratory and field evaluations have shown that thermosetting polymers (epoxy resins polyesters and polyurethanes) are not suitable sealants for pavements. Thin top coats, 5 to 20 mil, of epoxies and polyesters were found to be ineffective due to pinholes and wear resistance. Thick top coats, 1/8 in. to 1/4 in., made by incorporating sand with the polymer is expensive and haven't performed satisfactorily in severe climates. Only a few polyurethane systems have been evaluated. Some of the new polyurethane systems recommended by manufacturers do show promise; however, they are expensive when used as a surface coating. A 60- to 100-mil coating of polyurethane was recommended by the different manufacturers. An adhesive prime coat to bond the polyurethane to the pavement and to prevent pinholes from forming in the cured polyurethane is needed. Skid resistant materials would then have to be broadcasted on the polyurethane coating before it cures. Material cost would range from approximately $5.00 to $10.00 a square yard.

Asphaltic sealants, silicones, and chlorinated rubbers were found to be unsatisfactory as pavement sealants. Linseed oil is presently being used by most highway departments, but tests have shown that the material is not needed for proper air-entrained concrete and the protection is only temporary. Sheet membranes are difficult to apply and are expensive. The membranes also have to be coated with asphaltic concrete. There is not enough available information to conclusively confirm that these membranes really work.

Internally sealing concrete pavements with polymers is the most promising method of sealing pavements (on grade). Investigations of internally sealing concrete pavements by addition of minute particles of waxes at the time of mixing were made. Laboratory tests of this method of sealing pavements do show promise; however, field evaluations are needed before any conclusion can be reached.

15.15.9 Ceramic Materials. Bauxite is a generic name for the ore of aluminum, trihydrate and monohydrate with various impurities such as iron hydroxides and oxides and some minor silica. Calcined bauxite consists principally of corundum, sometimes called alpha alumina, which is a hard material of high elastic modulus. Calcined bauxite is expensive but less expensive than synthetic pure alumina, and also more useful in highway systems because the impurities form a glassy bond to the alumina crystals, and the differential strength and hardness of the alumina and the bond cause the calcined bauxite to wear uneven by under traffic and maintain rough surfaces which protect the road surface from rapid and contribute to skid resistance. Calcined bauxite can be used as all or part of the sand in a deeply textured concrete surface, where it contributes to the wear resistance of the surface. It can be used as chippings in asphaltic concrete surfaces, and it can be scattered as 3mm or 5mm grit in epoxy surfaces in bridge or road repairs. In a properly designed pavement system where the subbase and lower portion of the pavement is protected from saturation, calcined bauxite in the top of the wearing course will prolong the zero maintenance life of a pavement.

The Transport and Road Research Laboratory of the Ministry of Transport in England appears to have done the most work of any agency with calclined bauxite. Performance on roads has been related to high corundum content and to a texture in which the corundum crystals ranged in size from 5mm to 70mm and were bonded by a less abrasion resistant glassy matrix apparently consisting of silica and iron oxide. Neville* states that two scales of microtexture affect resistance to wear. The scales are 100mm to 250mm and smaller than 10mm. The coarser scale determines the general smoothness of the surface. This texture depends on the extent of traffic abrasion and whether differential wear or weathering has taken place. Fine microstructure is superimposed on higher areas of coarse microstructure. Texture is affected by polishing, abrasion, differential wear, and weathering; the range of hardness of

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*Reference 23, Chapter 12.

the minerals in the aggregate and their susceptibility to weathering largely determine the endurance of the surface.

Calcine bauxite appears to be the most promising ceramic material for consideration as a candidate zero maintenance material. Its use solely as an aggregate in or on conventional paving materials apparently will improve the performance of these materials but not to the point of their being a zero maintenance material. The use of calcined bauxite in combination with other candidate zero maintenance materials may be beneficial and should be given further consideration.

15.15.10 Prestressed Concrete Pavements. Based on the results of the available literature as cited in this report, the following conclusions appear warranted:*

a. The permanent compressive forces applied in prestressed slabs make it possible to build pavements continuous and eliminate thereby a great number of joints. Prestressing also provides a spring effect which will restore continuity in the event cracking in the slab sould take place due to accidental reasons such as extreme thermal stresses, shrinkage, or excessive loads.

b. The increased strength of prestressed concrete pavements points, of course, to the fact that such pavements can be produced with a thinner concrete section than conventional concrete pavements. They should be less susceptible to differential thermal conditions, which create severe stress conditions in thick pavements.

c. Cracking is greatly reduced, thereby restricting the amount of water reaching the subgrade and reducing the number of problems associated with subbase design.

d. Although cracking may occur, cracking in a prestressed concrete section does not represent failure. The cracked section is still able to resist increasing loads until the limit condition is reached, similar to that occurring in reinforced concrete slabs.

e. Pavements, if highly prestressed, would carry loads on average subbases with considerable deflections without detriment to the concrete slab but definitely causing permanent deformations in the supporting material.

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*References 1, 17, and 21, Chapter 13.

f. Joints cannot be eliminated completely. The joints which are required will necessitate some ingenuity in design and construction techniques.

g. Challenging design problems will involve determination of: (1) best method of prestressing, (2) amount of prestress required, (3) degree of prestress loss, (4) optimum type of materials, and (5) required pavement thickness.

h. Irregular pavement sections to be formed at intersections, pavement widenings, etc., together with horizontal and vertical curves will present design and construction details which will not always be solved easily.

i. Damage to such pavements, especially to their prestressing elements, will be more detrimental and will call for more elaborate repair procedures.

j. Prestress pavements will not necessarily allow acceptance of a weaker subbase than those presently used. Although research may eventually prove such a premise, sufficient information is not now available to support this concept.

The prestress concrete pavement system could conceivably be used with other systems and materials subject to some restrictions. Vacuum processing (Chapter 14) could be used in conjunction with prestress concrete to apparent advantage. The combination of prestress and VPC has the potential of providing an excellent paving system. Other materials and procedures such as those described in this report could be used in prestress concrete with the restriction that any material used should not increase the creep or thermal expansive properties of the concrete.

Apparently prestressed concrete pavement has progressed to the point where individual states are beginning to construct prototype sections. The Mississippi Highway Department recently placed five miles of prestressed pavement on US Highway 84 near Brookhaven, Mississippi.* The major problems remain in the area of joint construction, but the outstanding example of prestressed highway pavement in the United States, in Pennsylvania, appears to be very successful.

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*Reference 26, Chapter 13

Plans are to measure appropriate properties on the Mississippi prototype section such as subgrade friction, prestress loss, creep, etc. In view of the above information, and the promising outlook in prestress concrete pavement, it would appear desirable to establish a category to monitor progress thereon. Effort should be made to evaluate the Mississippi pavement with respect to zero maintenance requirements. Also, correspondence should be initiated to determine the behavior and condition up to the present time of the prestress pavement installations in the United States and abroad.

15.15.11 Vacuum-Processed Concrete. Based on the recent experience in Northern Europe on pavement-type strucrures and the claims of greatly improved physical properties, vacuum concrete would appear to have merit as a zero-maintenance material. The primary obstacle to successful prototype use would appear to be the limited rate at which the vacuum process can proceed behind a paving operation. Rates of approximately 2000 sq ft per hour have been achieved in Europe.[*] prove feasible for paving operations, rates of at least 10,000 sq ft per hour would probably be required. Recognized authorities[20,23] believe the process could be expanded by use of additional mats, more vacuum pumps, and possibly reduced vacuum time to accomplish the desired rate of movement. Use of set modifying or water-reducing admixtures would be compatible with VPC, however, the effect of these materials would probably be partially or wholly negated by the vacuum process. The throxotropic hardening would be induced long before the effects of the basic setting process, and the amount of water removed by vacuum would likely not be as great with mixtures employing water-reducing admixtures. Use of very fine material should be investigated before consideration for inclusion as a mixture to be vacuum processed. Since the prestressing process required high strength concrete, VPC might prove to be especially beneficial for prestressed slabs (Chapter 13). VPC would appear to be unsuited for use with polymer impregnation (Chapter 10) since the vacuuming process consolidates and densifies the material very thoroughly near the surface. Use with polymer-portland cement concrete (PPCC) would appear feasible, however, it is unlikely that the

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*Reference 23, Chapter 14

beneficial effects of both systems would be additive. Cross* has brought into perspective the situation at present with respect to vacuuming processing of bridge decks:

> "For many jobs the reduced water/cement ratio resulting from processing permitted a saving in cement. On still others elimination of delay in final finishing operations, due to the more rapid set of the concrete substantially reduced labor costs. Now, however, the pattern is changing and original cosntruction cost is taking second place to long-range durability in the minds of highway administrators. The penalties for bridge deck deterioration are very high, not only in maintenance expenditure, but in traffic delay and poor public relations. So high, in fact, that very substantial increases in the original cost of that so vital few inches that constitutes the measuring surface of bridges are now considered reasonable. VPC now has an opportunity to prove its worth as a way of preparing quality as well as a way of saving time and cement."

The same could be said for VPC applied to highly trafficked highways.

Texturing of pavement surfaces may be a problem if the texturing is to be done to the unhardened doncrete. Burlap texturing almost certainly could not be accomplished. Texturing by roll-in grooves or steel tines may still be possible, but timing during the operation would be very critical. Sawed texturing after the concrete has hardened should not be a problem.

15.15.12 Other Materials. Within the scope of this study, only a limited number of materials and materials systems could be examined. There undoubtedly are many other materials that might also be identified as candidate materials for zero maintenance pavements. Upon completion of the study, it was requested that two additional materials be given a cursory examination and commented upon by the investigators. These two materials were epoxy asphalts and high-range water-reducing admixtures for use in concrete. The following brief comments are on these materials.

15.15.12.1 Epoxy Asphalts. Epoxy asphalt concrete is a combination of a graded mineral aggregate, asphalt cement, epoxy resin, and a curing flexibilizing agent. The epoxy resins are classified as thermosetting plastics which means that once hardened by heat the epoxy resins cannot

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*Reference 11, Chapter 14

be softened and remolded by further application of heat.[1] The epoxy resins form the structural network for improving the strength properties of an asphalt. Epoxy asphalt concrete mixtures develop strength values that are several times higher than conventional asphalt concrete values.[1,2,3] Marshall stabilities for an epoxy asphalt concrete may be as high as 19,000 lb or ten times higher than conventional asphalt concrete. Although an epoxy asphalt will produce high strength characteristics, the cost may be five times the cost of a conventional asphalt.[3]

Epoxy asphalts were developed by a major oil company specifically to withstand high temperature jet blast and fuel spillage and to provide high load-carrying ability. In the early 1960's, epoxy asphalt concrete was placed at several air force bases. Field surveys of the epoxy asphalt installations indicated that the performance was satisfactory, but random cracking could be expected in epoxy asphalt concretes constructed in cold climates.[4] The survey also reported that blisters had been experienced on several installations. The blisters were probably caused by trapped vapors, and with proper construction techniques, the blisters could be reduced or eliminated.

The use of epoxy resins can improve many properties of an asphalt concrete, but because of the limited amount of available data on the random cracking potential in cold climates, epoxy asphalt cannot be recommended as a zero maintenance material unless further detailed study is made of this material.

15.15.12.2 High-Range Water-Reducing Admixtures for Concrete. High-range water reducers form a relatively new category of chemical admixture. They have been in commercial use in Germany since 1972 and in Japan since the late 1960's. However, development and trial periods pre-date their commercial use by as much as 5 to 8 years. In Germany and Japan, 5 to 6 million cubic metres of concrete containing this type of admixture have been placed. Their use appears to provide considerable advantages over conventional concrete and has consequently generated interest. The products have been variously known as "super plasticizers," "super admixtures," "high-gain water reducers," and "high-range water reducers." The preferable term is "high-range water reducers." They have been used for several purposes, including:

a. To reduce the water requirement of concrete mixtures by as much as 20 percent while permitting the workability (slump) to remain unchanged, and hence increase the strength.

b. To increase the workability by increasing the slump by as much as 6 in. while maintaining the same water-cement ratio, and hence strength.

c. To produce concrete with no change in slump or water-cement ratio (and hence strength) at lower cement content.

Work at the WES has revealed that these products can be used for these purposes. These results are highly desirable. However, there appears to be a detrimental side effect. This work on three of these admixtures reveals that compliance with frost-resistance requirements of CRD-C 87 (ASTM C 494) was not achieved. It appears that even though a proper air content was obtained in the freshly mixed concrete, using an acceptable air-entraining admixture, the bubble-spacing factor of the hardened concrete was unsatisfactory.

High-range water-reducing admixtures are not, by themselves, a zero maintenance material but when used in conventional concrete will improve many characteristics of that concrete. It is doubtful that the improvements it imparts to the concrete will be great enough to cause that concrete to become a material which will never need maintenance but the potential for deterioration of the concrete may be somewhat reduced. A recommendation that high-range water reducers be considered as a candidate for zero maintenance paving materials is not possible until more information becomes available about the performance of these materials. Additional research on these materials is warranted.

REFERENCES

CHAPTER 15

CONCLUSIONS AND RECOMMENDATIONS

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Table 15.1

Comparison of Characteristics of Asphalt Based Materials to Those of Asphalt Concrete

Characteristic	Gussasphalt	Asbestos Asphalts	Sulfur Modified Asphalts
Temperature dependency	NC*	NC	NC
Compressive strength	P-G	G	NA
Marshall stability	NA	NC	G
Flexural strength	NA	G	NC
Tensile strength	P-G	G	VG
Dimensional changes	NA	NA	NA
Durability	G	G	NA
Abrasion resistance	G	NA	NA
Toughness	NA	G	NA
Fatigue strength	NA	G	P-G
Dyanmic response	NA	NA	NA
Permeability	G	NC	G
Ease of construction	NC	NC	P
Ease of maintenance	NC	NC	NC
Environmental compatibility	NC	P	P
Availability	P	P	NC
Cost	P	P	P

* P = Poor.
NC = No change.
NA = Not available.
G = Good.
VG = Very good.

Table 15.2

Comparison of Characteristics of Special Concretes and Rigid Binder Systems to Those of Portland Cement Concrete

	Noncalcareous Inorganic Cements			Polymers in Concrete		
Characteristics	Sulfur Concrete	Sulfur Infiltrated Concrete	Phosphate Cements	Polymer Impregnated Concrete	Polymer-Portland Cement Concrete	Polymer Concrete
Compressive strength	NC*	VG	NC	G-VG	P-NC	G-VG
Flexural strength	NC	NA	NC	G-VG	G	VG
Tensile strength	NA	NA	NC	G-VG	G	G-VG
Modulus of elasticity	NC	G	NC	NC-G	NA	NA
Dimensional changes	P	NA	NC	P-NC	P-NC	P
Durability	P	G	NA	G	VG	G
Abrasion resistance	NA	NA	NC	NC	G	NA
Toughness	NA	NA	NC	NC	G	P-G
Impact resistance	NA	NA	NC	NA	NA	NA
Fatigue strength	NA	NA	NC	NA	NA	NA
Dynamic response	NA	NA	NC	NA	NA	NC
Permeability	VG	VG	NC	VG	P-G	VG
Ease of maintenance	NC	NC	NC	P	NC	NC
Ease of construction	NC	P	P	P	NC	P
Environmental compatibility	P	P	P	P	NC	NA
Availability	NC	NC	P	NC	NC	NC
Cost	NA	NA	P	P	P	P

* P = Poor.
NC = No change.
NA = Not available.
G = Good.
VG = Very good.

(Continued)

Table 15.2 (Concluded)

Characteristic	Expansive Cement Concrete	Fiber Reinforced Concrete	Ceramic Materials Calcined Bauxite	Prestressed Concrete**	Vacuum Processed Concrete	Sealants Internally Sealed Concrete
Compressive strength	NC	NC-G	G*	--	G	NC
Flexural strength	NC	VG	G	--	G	NA
Tensile strength	NC	NC-G	NC	--	G	NA
Modulus of elasticity	NC	NC	NA	--	NA	G
Dimensional changes	NC-G	NC	NC	--	G	NA
Durability	P-NC	NC-G	NC	--	G	NA
Abrasion resistance	NC	G	G	--	VG	NA
Toughness	NC	VG	NA	--	NA	NA
Impact resistance	NC	VG	NA	--	NA	NA
Fatigue strength	NA	G	NA	--	NA	NA
Dynamic response	NC	NC	NA	--	NA	NA
Permeability	G	NC	NC	--	G	G
Ease of maintenance	NC	P	NC	P	NC	NC
Ease of construction	NC	P	NC	P	P	P
Environmental compatibility	NC	NC	NC	NC	NC	NC
Availability	NC	NC	P	NC	P	NC
Cost	NC	P	P	NC	NC	P

** Comparisons not possible.

APPENDIX A

NOTATION

A.1 CHAPTER 7

d = least lateral dimension of specimen

g = aggregate content of mixture

h = height of specimen

K = constant

n = exponent

P = concrete strength

P_6 = strength of a 6-in. cube

R = rate of loading

S = strength at a given rate of loading, R psi/sec

S_1 = strength at a rate of 1 psi/sec

S_c = shrinkage of concrete

S_p = shrinkage of neat paste

V = volume of specimen

A.2 CHAPTER 9

A, B = constants

C' = cracking index

d = fiber diameter

E = modulus of elasticity

E_c = modulus of elasticity of composite

E_f = modulus of elasticity of fiber

E_m = modulus of elasticity of matrix

ℓ = fiber length

ℓ_c = critical fiber length

n = Ef/Em = modular ratio

p = percent volume of fibers

V = volume of one fiber

V_f = volume fraction of fibers

α = factor relating extreme fiber stress, the elastic section modulus, and applied moment

β = effective length factor

γ = factor relating direct tensile strength and modulus of rupture

ε_{np} = maximum total strain under cyclic loading

λ = efficiency factor for fibers of random orientation

λ' = total efficiency factor which includes λ and necessary fiber anchorage length

μ = Poisson's ratio

μ_f = Poisson's ratio for fiber

σ_{cr} = modulus of rupture for composite

σ_{cf} = direct tensile strength of composite

σ_{fc} = fracture strength of composite

σ_{fu} = fracture strength of fibers

σ_{mr} = modulus of rupture of matrix

σ_{mt} = direct tensile strength of matrix

τ = interfacial shear stress

ψ = correction factor to account for stress distribution on end portions of fibers

A.3 CHAPTER 10

A, B = constants

dt = incremental time

$d\sigma$ = incremental stress

S = strength of polymer-impregnated concrete

S_d = strength of aggregates

S_m = strength of cement paste

T_g = glass transition temperature

V_d = volume of aggregate

V_m = volume of cement paste

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