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16. Abstract

This document is a State-of-the-Art report surveying many new, as well as infrequently used techniques for improving the performance of the components of pavement systems. The goal of this work was to identify important factors affecting the design of new structural systems for "zero-maintenance" pavements. The three basic components of the conventional pavement system - the subgrade, the subbase and base, and the surface course - have been examined. The latest techniques of strengthening and stabilizing the subgrade component, and preserving its improved properties are reviewed. The feasibility of utilizing synthetic aggregates and waste products in "zero-maintenance" pavements is addressed. The utilization of new materials and new systems such as prefabricated panels and pile supported pavements have also been investigated. Evaluations of existing information on the individual pavement system components have been synthesized to produce one modified conventional, and seven new structural pavement systems that may be capable of satisfying "zero-maintenance" criteria.

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TABLE OF CONTENTS

PAGE

PAGE

LIST OF TABLES

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

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CHAPTER 1

INTRODUCTION

This report forms 'Task A' in the work statement of the project study entitled "New Structural Systems for 'Zero-Maintenance' Pavements". Task A is a "state-of-the-art" report presenting a survey of the technology and identifying the unique and infrequently used concepts and systems suitable for "zero-maintenance" pavements. The primary objectives of the total research contract are the following:

1. Assess potential of innovative structural concepts and systems to serve as "zero-maintenance" pavements; and

2. Provide pertinent documentation on design, construction and detailed investigation of the more promising systems.

The entire research work is divided into Tasks A through G. This report relates to the fulfillment of Task A only, which involves identifying potentially innovative structural systems and a preliminary selection of the promising concepts.

For this study, a "zero-maintenance" pavement is defined as one that will provide at least 20 years of maintenance-free operation on a high-volume expressway and at least 10 more years with routine maintenance before rehabilitation is required.

It is assumed that the loads of the zero-maintenance pavements will be within the limits of the National Cooperative Highway Research Program study of 1973 (Whiteside et al., 1973)[1].

The scientific approach to the solution of an engineering program involves first defining its scope and functional purpose, and then seeking solutions to the problem within the scope and purpose defined. Having defined the zero-maintenance pavement and the objectives of the proposed research, one can argue that pavement design is not necessarily governed by the weight of a vehicle; rather, the purpose of a pavement is to provide a functional surface for the safe operation of a vehicle. The dynamic interaction of vehicle vibration and the environmental effects on the pavement are the primary considerations in judging the functional purpose of a pavement system. This analogy

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Whiteside, R.E. et al. (1973) "Changes in Legal Vehicle Weights and Dimensions - Some Economic Effects on Highways," National Cooperative Research Program Report No. 141, Highway Research Board.

represents a departure from the accepted pavement design analysis, wherein the weight of the vehicle is the primary design criterion.

The state-of-the-art report is organized with the following points of view. Most pavement structures fail because the subgrade is weak either initially or is made weak due to environment changes, causing excessive deformations of the subgrade. It is, therefore, necessary to search for methods that will improve the natural subgrade and maintain its properties during the lifespan of a zero-maintenance pavement. Consequently, the first effort was to investigate the unique and infrequently used methods for improvement of subgrade soils. Conventional methods, e.g., lime stabilization, have not been included. Secondly, the bases and subbases have been examined, and finally, the new concepts involving the top surface have been examined. After reviewing the available literature, an effort has been made to put together the overall pavement system using either some modifications or completely new schemes. The study is presented in the following sequence:

Chapter 2 - Describes the problem soils and the innovative concepts to strengthen them.

Chapter 3 - Identifies new concepts and materials that can be used in subbase and base course.

Chapter 4 - Surveys innovative composites that can be used Additionally, new concepts and systems of pavements are identified and evaluated.

Chapter 5 - Selects five structural systems that show great promise. Many subsystems can be evolved from these five systems.

Chapter 6 - Summarizes results and provides recommendations for future work.

CHAPTER 2

THE SUBGRADE

2.1 INTRODUCTION

Traditionally, a subgrade is defined as the foundation of a pavement structure that usually consists of the surface course with or without additional base and/or subbase courses. The subgrade may be a natural soil exposed in a cut or an earth embankment built on natural soils. The design and construction of an embankment as a roadway subgrade is an important and complex but, nevertheless, relatively standardized process. All roadway embankments must be satisfactorily compacted in uniform layers to insure adequate stability and small and uniform settlement. In most instances, however, the governing factor in the design and construction of a subgrade is the condition of the natural soil in-place. This chapter concentrates on the techniques of treating or stabilizing the in-situ soils for use as the subgrade of a pavement or the foundation of a roadway embankment.

The subgrade, or the in-situ soil, must be designed to withstand the effects of the repetitious nature of the traffic loading in addition to the surcharge weights of the pavement and the embankment. Furthermore, the in-situ soil beneath the pavement and the embankment must withstand the deteriorating effects of climatic and environmental factors such as freezing and thawing, moisture variations, and chemical changes. An ideal subgrade must be able to maintain the permanency of its design properties and to carry out its design functions with extremely small deflection and without requiring any maintenance.

In order to design an ideal subgrade, special stabilization techniques have to be employed, especially when problematical soils are encountered. Table 1 presents several major types of soils that exhibit special problems in pavement design. The problems are usually either low strength or high volume change characteristics, or both, resulting from loading, climatic effects, or environmental changes. The stabilization techniques may, therefore, be grouped into three general categories: densification; strengthening; and protection against environmental changes. Table 2 is a brief presentation of the stabilization techniques that may be used in highway design and construction.

This chapter summarizes the results of our review of these special stabilization techniques with particular emphasis on their application to pavement designs and construction. The following sections begin with a description of the stabilization techniques related to densification, followed by techniques related to strengthening and protection. Finally, the techniques are summarized to show their adaptability to highway construction. A concept regarding building a zero-maintenance subgrade and/or subbase and base using the conventional pavement system is recommended.

TABLE 1

SUBGRADE MATERIALS REQUIRING SPECIAL STABILIZATION TECHNIQUES

TYPE OF MATERIAL	MAJOR PROBLEMS
Loose Sand	High compressibility, especially under repetitious loading
Soft Clay	Low strength and high compressibility
Silt and Silty Fine Sand	Strength and compressibility very sensitive to moisture variation
Expansive Soil	High volume change characteristics under varying moisture conditions
Organic Soil	Very low strength and very high compressibility
Frost Susceptible Soil	Weakening and swelling under repeated freeze-thaw cycles
Collapsible Soil	Large deformation as a result of saturation
Sensitive Clay	Weakening under mechanical disturbance and environmental change
Refuse Fill	Erratic in strength and compressibility characteristics

TABLE 2

SUBGRADE STABILIZATION TECHNIQUES

PURPOSE OF STABILIZATION	STABILIZATION TECHNIQUES
Densification	Dynamic Consolidation Vibroflotation Terra-Probe Compaction Piles Compaction Grouting Blasting
Strengthening	Vibro-Replacement Chemical Injection Other Methods
Protection Against Environmental Factors	Membrane Encapsulation Insulation Drainage

2.2 DENSIFICATION

2.2.1 Dynamic Consolidation

Dynamic consolidation is a technique that improves the mechanical characteristics of a weak soil by repeated application of a very high intensity impact energy to the surface. The procedure consists of dropping a weight of 5 to 40 tons (4.54 to 36.3 MT) from a height of 20 to 120 ft (6.1 to 36.6 m) on the ground to be densified. The reduction in void ratio causes the strength and bearing capacity to increase while compressibility is greatly reduced. With careful phasing of the tamping process and energy levels, dynamic consolidation has successfully treated low permeability, unsaturated and saturated silty and clayey soils, as well as virtually any granular soil and refuse fill. In general, the process has been found to be especially suited to large open sites on reclaimed land, fills, and alluvial soils. The technique is effective to a depth as much as 100 ft (30.5 m), and the improvement may be 2 to 10 fold.

The technique was first described by Lewis (1957) and Parsons and Toombs (1968) using relatively low impact energy. In recent years, Menard (1972, 1975) has used impact devices with an extremely high energy of impact to compact soils to a depth many times the diameter of a wide impact mass. It has been found that compaction depth depends very markedly on the impact energy available per unit area of contact, which often exceeds 50,000 ft-lb/ft^2 (7.3 x 10^5 J/m^2). The shock wave and high stresses induced in the ground result in (1) compression of micro-bubbles in the saturated soil, or compression of air-filled voids; (2) liquefaction under repeated impact; (3) creation of fissures and preferential drainage paths; and (4) rapid reconsolidation (Menard and Broise, 1975).

The operation is usually carried out in a number of phases, sometimes several weeks apart, to allow dissipation of induced excess pore pressure. For a typical site where soil properties are to be improved at great depth, spacing for the initial phase of impact is 15 to 40 ft apart. Closer spacings are used in the subsequent phases to compact the soils at shallower depths. The effects of a series of dynamic consolidation passes on the soil properties are shown on Figure 1.

Figure 2 shows field measurements of the variation of shear strength and settlement with time after the treatment of a 20-foot-thick (6.1 m), nonstratified, soft plastic clay by dynamic consolidation (Menard and Broise, 1975). The shear strength of the untreated clay was 12 to 20 kN/m^2, and immediate settlement of a similar embankment on untreated clay was 150 mm with no sign of stabilization.

Figure 3 presents Menard pressuremeter and cone penetrometer results of sand hydraulic fill placed on top of deltaic silt deposits before and after treatment by dynamic consolidation

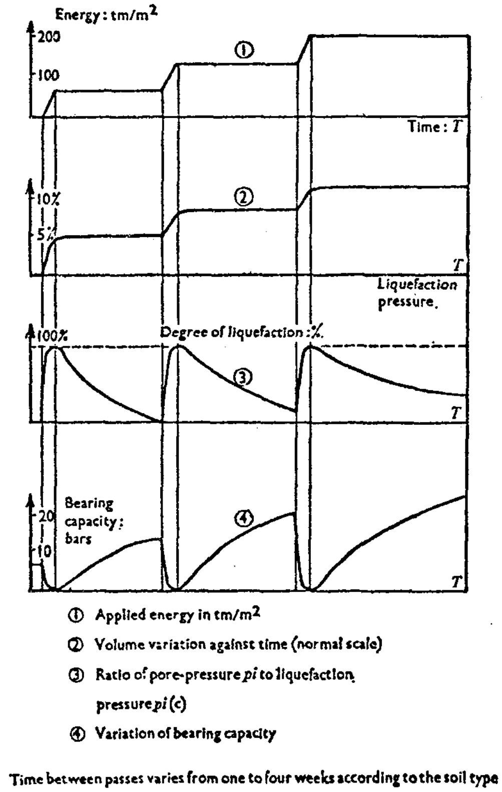
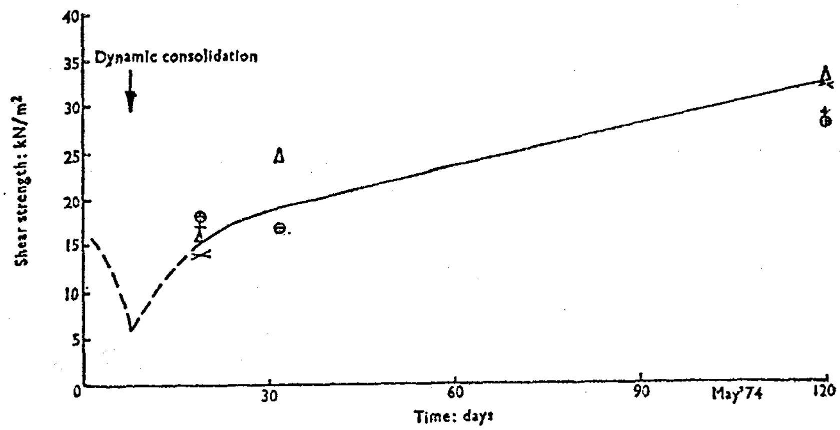


FIGURE 1 VARIATION OF A SOIL SUBJECTED TO SERIES OF DYNAMIC CONSOLIDATION PASSES (AFTER MENARD AND BROISE, 1975)



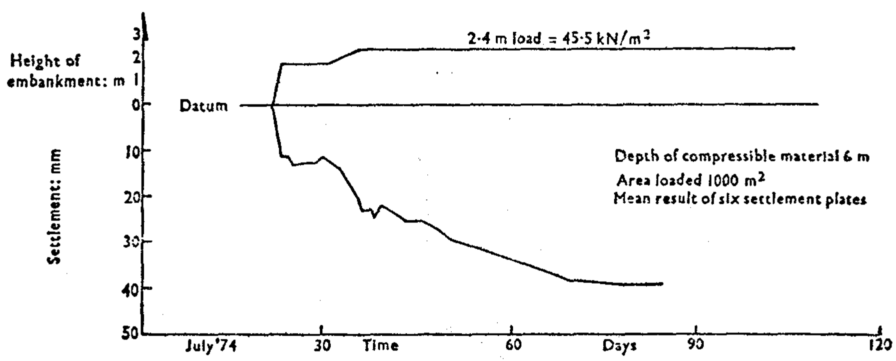


FIGURE 2 TEST EMBANKMENT – SHEAR STRENGTH AND SETTLEMENT (AFTER MENARD AND BROISE, 1975)

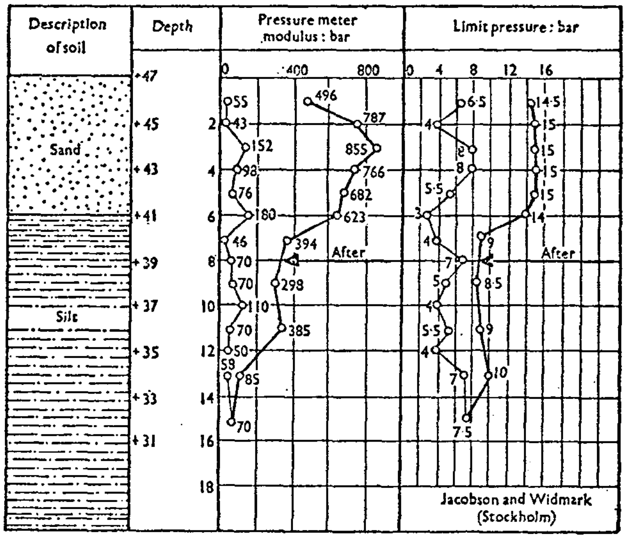


FIGURE 3(a) COMPARISON OF RESULTS OBTAINED WITH PRESSUREMETER: IMPROVEMENT OF PRESSUREMETER CHARACTERISTICS OF SOIL - MEAN VALUES FOR 20 BORINGS MADE ON SITE (AFTER MENARD AND BROISE, 1975)

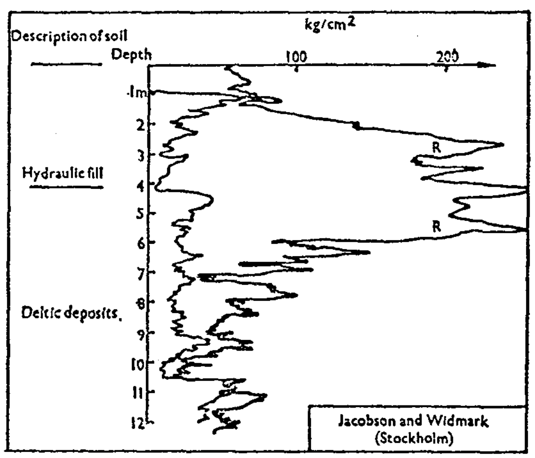


FIGURE 3(b) COMPARISON OF THE RESULTS OBTAINED WITH PENETROMETER: TYPICAL RESULTS OBTAINED FROM STATIC PENETROMETER BOREHOLES - INCREASE OF POINT RESISTANCE IN SANDY SILT IS DUE TO DYNAMIC CONSOLIDATION (AFTER MENARD AND BROISE, 1975)

(Hansbo et al., 1973). The site was to be used to construct three 20,000 m^2 warehouses to sustain a stocking load of 10 t/m^2 at an allowable differential settlement of less than 1 cm over a 10-m distance. The site was dynamic consolidated with a mean energy of 240 tm/m^2. The improvement in shear strength and deformation moduli is evident on Figure 3. The deformation moduli as measured by the Menard pressuremeter increased by more than 500 percent for the first 7 m and more than 200 percent at a 15-m depth. The point resistance, as measured by the cone penetrometer, increased from 2 to 10 fold above a 10-m depth.

The dynamic consolidation process and some of its instrumentation developed for its control are covered by patent and trade names. The technique has gained rapid acceptance in many countries, especially in Europe. Table 3 summarizes the results of several examples of the application of the dynamic consolidation technique.

The site to be consolidated must first be prepared to support the weight of the tamping machine, which may range from 60 to 2000 tons (54.4 to 1814.4 MT). When the site soils are sensitive to water, the operation must safeguard against bad weather by providing proper surface drainage. Water rising to the surface during the consolidation process must also be removed by means of peripheral trenches or drains. Fills are usually required to fill in the prints created by the impact.

The performance during the operation is usually evaluated with cone penetrometer and Menard pressuremeter to measure the change of strength and compressibility over a minimum period of 3 to 4 weeks. The evaluation is usually aided by numerous piezometers, nuclear density-moisture tests, and topographic measurements.

The vibration produced by the impacts is relatively large and may prohibit the employment of this technique in urban areas. Menard and Broise (1975) investigated the effects and modes of vibration that resulted from the impact of dynamic consolidation. The usual frequencies of the vibrations caused by the tamping vary between 2 and 12 Hz and appear to be transmitted by the substratum; the most frequent value is on the order of 3 to 4 Hz. The wave velocity (Rayleigh wave) is very low in the zone liquefied by tamping, and increases as it moves away and becomes normal at a great distance. The wave train is weakly dampened and comprises 3 to 6 waves of almost constant amplitude. At 30 m from the point of impact, the vertical and horizontal velocities of the soil particles remain much below the value of 5 cm/s admissible as an acceptable limit for a dwelling construction. The amplitude of the vibrations is slightly influenced by the height of fall of the pounder, but increases noticeably with the area of impact.

TABLE 3

EXAMPLES OF DYNAMIC CONSOLIDATION APPLICATION

PROJECT	SOIL TYPE	AREA TREATED	DEPTH TREATED	IMPACT ENERGY	RESULTS	COMMENT	REFERENCE
Warehouses	Hydraulic fill over silt and clayey sand	25 acres	50 ft.	160 k-ft/ft^2, 12-ton tamper, drop 40 ft. 3-7 passes	Cone Resistance incr. 2-10 folds to max. over 300 psi Deformation Modulus incr. 2-8 folds to max. of 11,000 psi	Carried out in 6-month period through winter	Hansbo, et al. (1973)
Oil Storage Tanks	Dredged sand and gravel over organic silty clay and sand	13 acres	60 ft.	130-260 k-ft/ft^2	Deformation Modulus incr. 2-3 folds		Menard and Broise(1975)
Sewage Disposal Works	Soft plastic clay	9 acres	20 ft.		Shear strength incr. from 300 to 600 psf in 120 days		Menard and Broise(1975)
Highway Interchange	Organic silt and peats (moisture content 150-1200%)	20 acres	20-45 ft.	400 k-ft/ft^2 7 passes	Settlement of the 23 ft. high embankment was 1.2 inch in 113 days	Sand and gravel fill used after each impact pass	Menard and Broise(1975)
Slipway Foundation	Waste coal fill	2 acres	30 ft.	20-60 k-ft/ft^2 14-ton tamper drop 30-80 ft	Deformation Modulus incr. 3 times to 3500 psi	Surface Settlement 0.5-1 m during compaction	West and Slocombe(1973)
Storage Building for Steel Billets	Rubble fill over soft silty clay with sand pockets and seams	2.5 acres	20-30 ft.	14-ton tamper 60 ft max free fall 3 passes	Deformation Modulus incr. 50-200% to 1500 psi in the silty clay and 2500 psi in the fill. Modulus of subgrade reaction was 40-70 psi on 77 ft^2 plate after treatment	Completed in 8 working weeks	West(1975)

1 acre = 4047 m^2 1 k-ft/ft^2 = 1489.5 kg-m/m^2
1 ft = 0.305 m   1 psi = 0.0703 Kg/cm^2
1 in = 2.54 cm   1 ton = 0.907 metric ton

TABLE 3 (continued)

PROJECT	SOIL TYPE	AREA TREATED	DEPTH TREATED	IMPACT ENERGY	RESULTS	COMMENT	REFERENCE
Aluminum Foundry	Marl fill over soft silty clay		10 ft.	14-ton tamper varied height of free fall	Deformation Modulus incr. 3 folds to an average of 1000 psi in 150 days		West(1975)
Oil Storage Tank Farm	Silty sand hydraulic fill	50 acres	60 ft.	90 k-ft/ft^2, 40-ton tamper, 110' drop, 4 passes, 6 impacts per point. Impact points 40 ft centers	Rel. Density incr. from 50-60% to 75%	Est. cost \$1.70/ft^2 (1975) vibration from impact 3 in/sec at 40 ft distance, negligible at 100 ft.	Dames & Moore Project(1977)

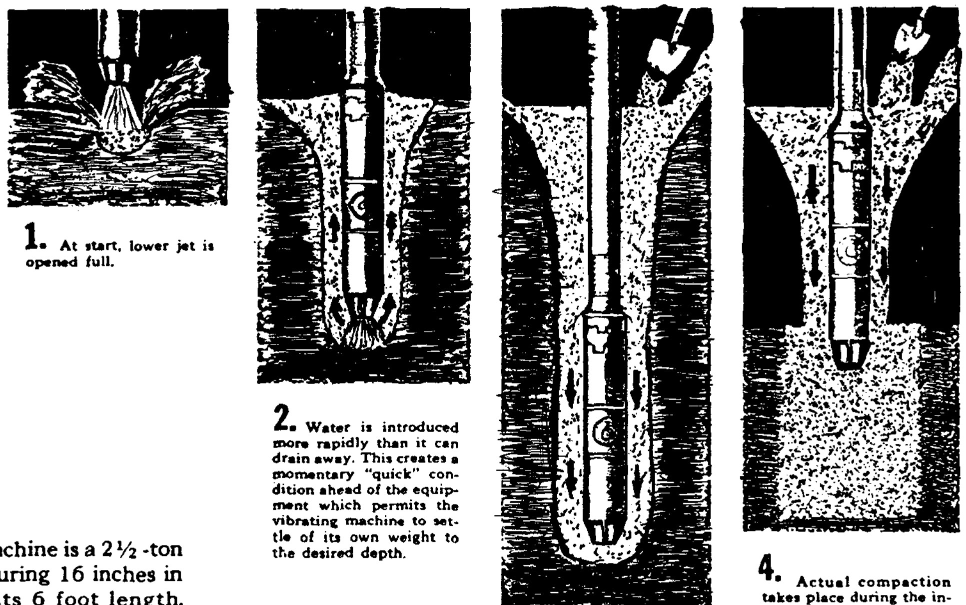
2.2.2 Vibroflotation

The principle of densification of base sand deposits by vibroflotation was first described in 1936 by Steuerman in a Russian journal. The first application of the method for the improvement of building foundation soils was made shortly thereafter in Germany; it was introduced into the United States in 1939. Through the years, the vibroflotation method has proved to be a viable means for densifying relatively deep deposits of loose, cohesionless sand. Mitchell (1970, 1973) and Greenwood (1970) summarized the state-of-the-art of the application of this technique in separate articles published in the early 1970s.

The vibroflotation process involves the sinking of a vibroflot probe through the zone of material to be densified. Water under pressure is usually introduced at the tip of the vibroflot in order to create a "quick" condition in the material below the tip to facilitate the sinking of the vibratory probe under its own weight. Actual compaction takes place during the intervals between the lifts that are made in returning the vibroflot to the surface. During the withdrawal process, sand is shoved in from the surface, and the water from the lower jet is transferred to the top jets to carry the sand to the bottom of the hole. During each interval between the lifts, the vibrator is allowed to operate until the desired density around the lower part of the vibroflot is obtained. The desired density is measured by the power consumption of the vibratory motor. When the measured power consumption reaches a predetermined amount, the probe is raised from 1 to 3 ft (30.5 to 91.5 cm), and the vibration and backfill process continues until the entire depth of soil is compacted. Figure 4 illustrates the vibroflotation process described above.

The vibroflot is a 2- to 2½-ton (1.81 to 2.27 MT) vibratory compactor measuring 16 in. (40.6 cm) in diameter over its 6- to 7-foot (1.83 to 2.13 m) length. A follower or adaptor section, varying in length depending on the depth of compaction, is attached to the top of the vibroflot, and the total assembly is suspended from a crawler crane (Figure 5). An eccentric weight inside the vibroflot develops a horizontal centrifugal force of about 10 to 18 tons (9.1 to 16.3 MT) at an 1800 to 3000 rpm vibration frequency with a power input on the order of 75 KW. The eccentric movement at the bottom is usually less than 1 inch (2.50 cm) during operation. Little information is given in the literature concerning the influence of the frequency and amplitude of vibration, the magnitude of the vibrating force, and the amount of power input on the degree and range of densification. However, it is believed that the larger the power input and the amplitude of horizontal vibration, the larger are the radius effects (Greenwood, 1970). A study has also shown that the amount of densification obtained increased with peak acceleration up to a value of about 0.5 g, beyond which point further increase was small (Mitchell, 1970).

At the spot to be compacted, the VIBROFLOT is jetted into the soil. The compaction sequence has four basic steps:



1. At start, lower jet is opened full.

2. Water is introduced more rapidly than it can drain away. This creates a momentary "quick" condition ahead of the equipment which permits the vibrating machine to settle of its own weight to the desired depth.

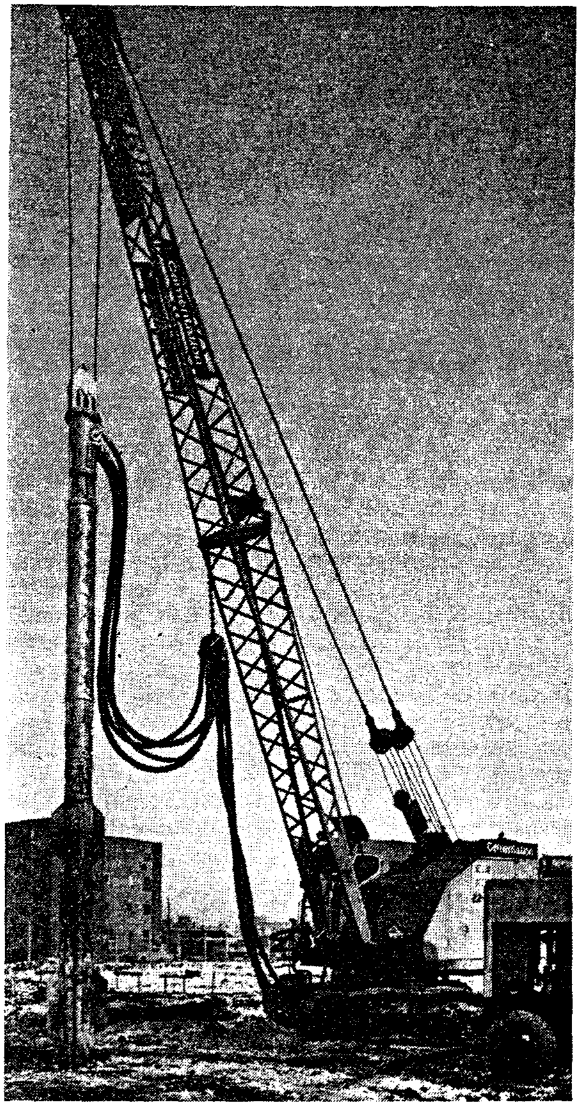
3. The water from the lower jet is transferred to the top jets and the pressure and volume is reduced just enough to carry the sand to the bottom of the hole.

4. Actual compaction takes place during the intervals between the one-foot lifts which are made in returning the VIBROFLOT to the surface. The vibrator is first allowed to operate at the bottom of the crater until the desired density around the lower part of the machine is attained. By raising the vibrator step by step and simultaneously backfilling, the entire depth of soil is compacted.

The Vibroflot machine is a 2½-ton compactor measuring 16 inches in diameter over its 6 foot length. The adaptor section shown will vary in length depending on depth of compaction. Ten tons of centrifugal force from the motor driven eccentric cause an average settlement of one inch of soil per foot of Vibroflot compaction.

1 in = 2.54 cm
1 ft = 0.305 mm
1 ton = 0.907 metric ton

FIGURE 4 COMPACTION PROCESS BY VIBROFLOTATION (AFTER MITCHELL, 1973)



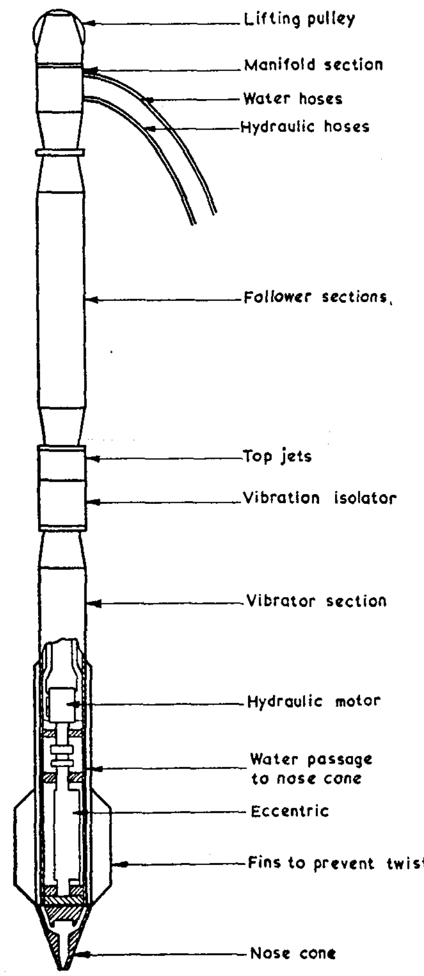


FIGURE 5 VIBROFLOT AND THE CRAWLER CRANE (AFTER GREENWOOD, 1970)

Water flow from jets at the top and bottom of the vibroflot is available at a rate of 60 to 400 gpm and at a pressure of 60 to 120 psi (42.2 to 84.4 T/m^2). It is almost always necessary to circulate water to achieve effective treatment. In highly permeable coarse-grained cobbles, the advantages of water circulation are negligible, and treatment may be undertaken without water. The presence of ground water does not generally eliminate the need of a water jet, nor does it affect the applicability of the method.

The soils most suitable for densification by the vibroflotation method are sands containing less than 10 percent of fines (<0.075 mm) and 10 percent of gravel (>4.76 mm). Figure 6 shows the most desirable range of gradation of soils for treatment by the vibroflotation method. Mitchell (1973) summarizes that the sinking rate of the vibroflot through the typical cohesionless soils is about 3 to more than 10 fpm (305 cm/min), while the withdrawal rate is about 1 to 2 fpm (30 to 60 cm/min). In clean, free draining soils, 3 cu ft to as much as 20 cu ft (0.08 to 0.56 m^3) of sand backfill will be required per foot of compacted depth, and a cylindrical column 8 to 10 ft (2.44 to 3.05 m) in diameter is compacted by one penetration of the vibroflot. The degree of compaction decreases radially from the center of the cylindrical column. Usual spacings of vibroflot holes are 7 to 8 feet (2.13 to 2.44 m), arranged in either square or rectangular patterns; however, the triangular pattern is preferred because it gives the greatest compaction effort overlap (D'Appolonia, 1953; D'Appolonia, Miller and Ware, 1955). The backfill material is usually 20 to 70 mm in size and frequently of uniform grading within this range.

Although the depth of treatment is usually less than 30 ft (9.14 m) on most projects, satisfactory densification of sand to depths greater than 62 ft (18.90 m) by vibroflotation has been reported (Steuerman and Murphy, 1957). Relative densities of at least 70 percent can usually be obtained at points midway between compaction; allowable bearing pressures subsequent to compaction are usually 2 to 3 tons/ft^2 (19.53 to 29.30 T/m^2). D'Appolonia, Miller and Ware (1955) present a procedure for determining the vibroflot spacing required to obtain a specified minimum relative density.

Mitchell (1970) summarized examples of vibroflotation applications in and prior to 1968 in the form of a table. Table 4 reproduces the summary, supplemented by applications published since 1968. It is seen from this table that all the applications are applicable to treatment of foundation soils for buildings and industrial facilities in areas limited in size. No such application has been published that is related to highway construction.

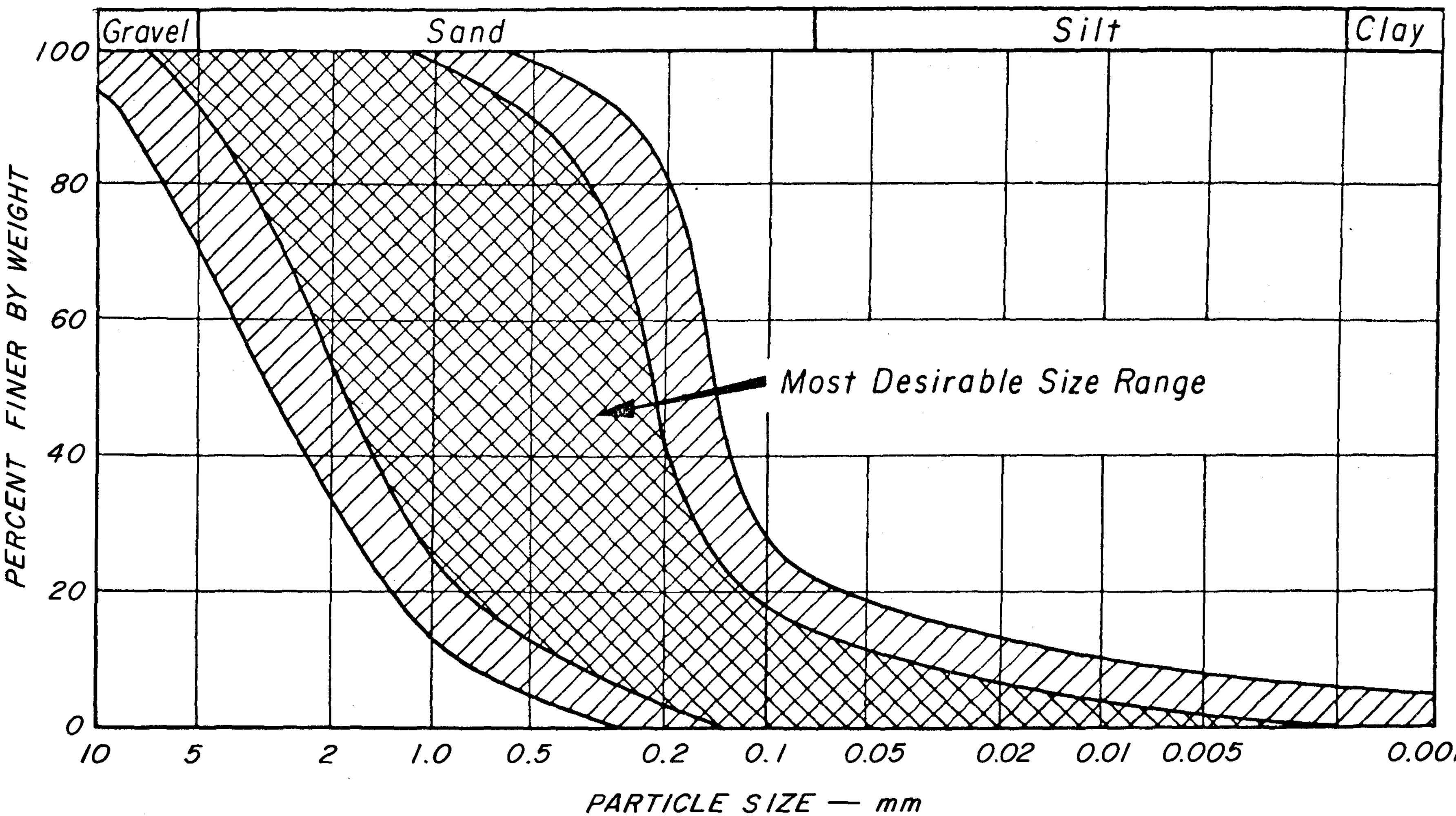


FIGURE 6 RANGE OF PARTICLE SIZE DISTRIBUTIONS SUITABLE FOR DENSIFICATION BY VIBROFLOTATION (AFTER MITCHELL, 1973)

TABLE 4

EXAMPLES OF VIBROFLOTATION APPLICATION
(Updated from Mitchell, 1970)

PROJECT	DATE	SOIL TYPE	DEPTH FT.	AREA TREATED FT.^{2}	SPACING OF INSERTIONS FT.	RELATIVE DENSITY		COMMENT	REFERENCE
						INITIAL	FINAL		
Building Foundation	1937	Sand	23.5	6202	6.5	43	80	Allowable bearing capacity increased from 1.5 to 3.0 tons/ft^2	Schneider (1938)
Foundation Support for Grain Silo	1940	Sand and gravel	15			63	85-95	Densified layer was 30-45 ft below surface. Used for pile support.	Cassel (1956)
Enders Dam	1948	Well graded sand (14% fines)	20		8	47	79	Unsatisfactory in silty material with 56% fines	USBR (1948)
Foundation Support for Kilns and Chimney	1948	Sand and gravel	12-16	9000	7-8	7-58	70-100		Fruhauf (1949) Westberg and Ireland (1972)
Foundation Support for Phosphate Plant	1951	Clean loose sand	12	156,000	7.5	33	78	Cost \$2.70/yd^3	D'Appolonia (1955)
Foundation Support for Wood Cellulose Mill	1952	Fine sand	16-20	175,300	6.5-8	0-40	75-92½	Little correlation between R_d by std. penetration tests and by samples	Mystkowski (1953)
Support for Hangar	1954	Gravelly sand and sandy gravel	20-31	-25,000		-0	-80		Cassel (1956)
Power Station Foundation	1954	Sandy gravel				33-80	85-95	Zones containing clay did not compact well	Cassel (1956)
Sand Island Fill for Tunnel	1957	Well graded sand	42-62		8-12			No correlation between R_d by std. penetration test and by samples	Steuerman and Murphy (1957)
Foundation Support for Power Station	1959	Glacial sand and gravel	17		5.5-7.5	40-60	85-90	\$1.80/yd^3 for zones specified at 70% R_d \$2.51/yd^3 for zones specified at 85% R_d	Petersen and Nestezenko (1959)

1 yd^3 = 0.764 m^3 1 ton/ft^2 = 9.765 T/m^2
1 ft = 0.305 m 1 psi = 0.0703 kg/cm^2
1 in = 2.54 cm

TABLE 4 (continued)

PROJECT	DATE	SOIL TYPE	DEPTH FT.	AREA TREATED FT.^{2}	SPACING OF INSERTIONS FT.	RELATIVE DENSITY INITIAL	RELATIVE DENSITY FINAL	COMMENT	REFERENCE
Foundation Support for Drydock	1961	Well graded fill	11	68,000	10	50	75		Tate(1961)
Foundation Support for 20-Story Building	1965	Loose sand	23		6.25	Loose	80	Cost \$1.90/yd^3. Eliminated an estimated 5" settlement	Grime and Carthey(1965)
Foundation Support for Building	1967	Loose fine sand with clay inclusions	20		5-7.5		to 80	No correlation between R_d by std. penetration tests and by samples	Woodward-Clyde and Associates Project Report
Support of Fertilizer Factory on Reclaimed Land	1968	Misc. fill, silts, sands clay	25		6-6.5			Stone column method used Cost 2/3 that for conventional piles	Luce(1968)
Sugar Silo Site	1968	Fine sand, fine silty sand, clayey sand, up to 30% clay	30	10 acres	7.5	400-1200 psi cone resistance	900-2700 psi cone resistance	Coarse aggregate backfill gave improved results in more clayey materials	Webb and Hall (1968)
Factory Site	1968	Windblown sand 10% < No. 200	20	5 acres	6	200-1000 psi cone resistance	1000-2000 psi cone resistance	0.33-0.70 yd^3 fill added per ft depth of penetration	Webb and Hall (1968)
Oil Tank Site	1968	Loose dune sand fill	12	tank perimeter				0.33-0.70 yd^3 fill added per ft depth of penetration	Webb and Hall (1968)
Building Foundations	1968	Loose sand under limestone (8-15' layer)	15-20	24,000	<7.5			Backfill consumption was 8.4 ft/ft. Building settlements less than 1/8 inch	Reed(1968)
Building Foundations	1967	Hydraulically filled medium to fine sand with clay inclusions	30		6-7	50-90	>75	Backfill consumption was 6-7 yd^3/ft. \$2.80-\$3.90 per yd^3 densified	Basore and Boitano(1969)

TABLE 4 (continued)

PROJECT	DATE	SOIL TYPE	DEPTH FT.	AREA TREATED FT.^{2}	SPACING OF INSERTIONS FT.	RELATIVE DENSITY INITIAL	RELATIVE DENSITY FINAL	COMMENT	REFERENCE
Graving Dock and Steel Production Facilities	early 1970	Hydraulically filled clean medium sand	35		8-12	600 psi cone resistance	2000-3000 psi cone resistance	The probe applied 18 tons of horizontal centrifugal force and an unrestrained horizontal vibration amplitude of 1.25 in.	Brown and Glenn (1976)

In all examples tabulated in Table 4, sand treated by the vibroflotation process attained a minimum relative density of 75 percent and a cone penetrometer resistance of greater than 1000 psi (70.3 kg/cm^2). Plate load tests performed on clean, relatively uniform, medium sands (Greenwood, 1970) have also shown significant improvement with regard to the settlement characteristics of the sand after vibroflotation treatment. Figure 7 presents the plate test results in terms of modulus of subgrade reaction, k, versus applied plate pressure. The magnitude of the modulus of subgrade reaction for the treated sands is generally 1.5 to 3 times higher than the untreated materials.

As an aid for preliminary design, Thorburn (1975) suggested relationships between achievable relative density and spacing of vibration and relationships between allowable bearing pressure for limited settlement of 1 inch (2.54 cm) and spacing of vibration; the relationships are shown as Figures 8 and 9, respectively. It should be appreciated that the response of cohesionless soils to treatment will vary from site to site and will depend on the particle size distribution of the soil, the presence of water table, and the amplitude, frequency, and penetration and withdrawal rates of the vibroflot.

2.2.3 Terra-Probe

In recent years, a new method known as "Terra-Probe" has been successfully used in the United States for compacting deep, loose sand deposits (Janes, 1973; Anderson, 1974; Brown and Glenn, 1976). Compaction by this method is accomplished by the use of a vibratory pile-driving apparatus together with an open-end tubular probe. The tubular probe is driven into and extracted from the soil by the vibratory energy produced by the vibrodriver, producing densification both inside and outside of the tubular probe. The concept and procedure used for the Terra-Probe densification are very similar to that described for the vibroflotation method, except that the vibratory action is vertical and the probe is extracted at a slow continuous rate. Similar to the vibroflotation process, water jetting is used during both the penetration and extraction operations.

To date, the vibratory pile-driving apparatus used have been in the frequency range of 720 to 1,100 cycles per minute with the normal operating frequency being 900 cycles per minute. The device has been able to create amplitudes of 3/8 in. to 1 in. (9.5 to 25 mm). The best probe material used has been an open-end 30-in. (760 mm) pipe of 3/8-in (9.5 mm) wall thickness with 4- to 6-in. wide (100 to 150 mm) 1/2-in. (13 mm) thick bands spaced 5 to 10 ft (1.5 to 3 m) apart on the outside of the pipe together with wider driving and clamping bands at the bottom and top of the probe. Field usages of other sizes of tubular probes have been accomplished but with less effect. Smaller diameters give less densification volume inside the probe while larger pipe requires

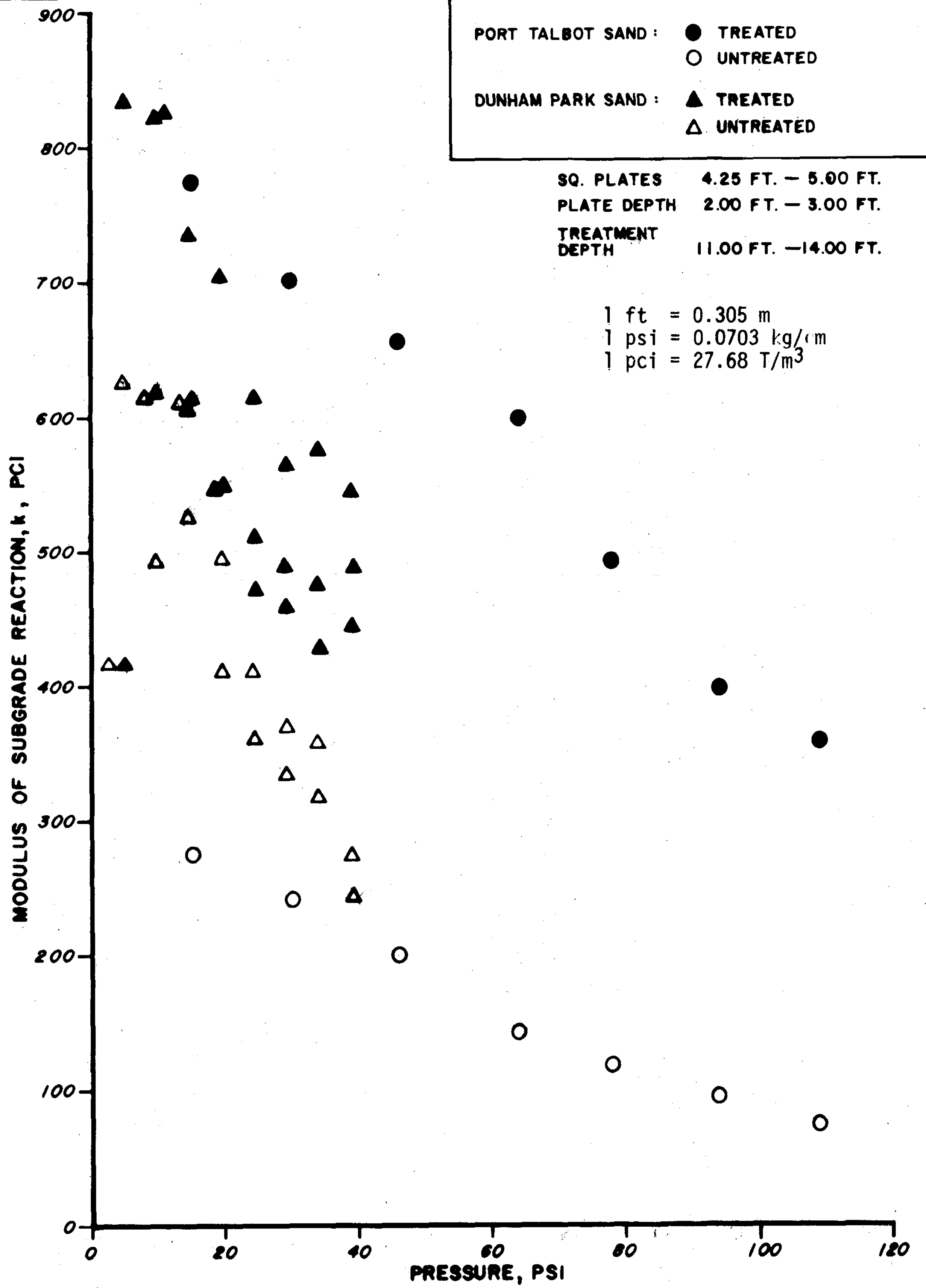


FIGURE 7 RESULTS OF PLATE LOAD TESTS ON SAND BEFORE AND AFTER VIBROFLOTATION TREATMENT

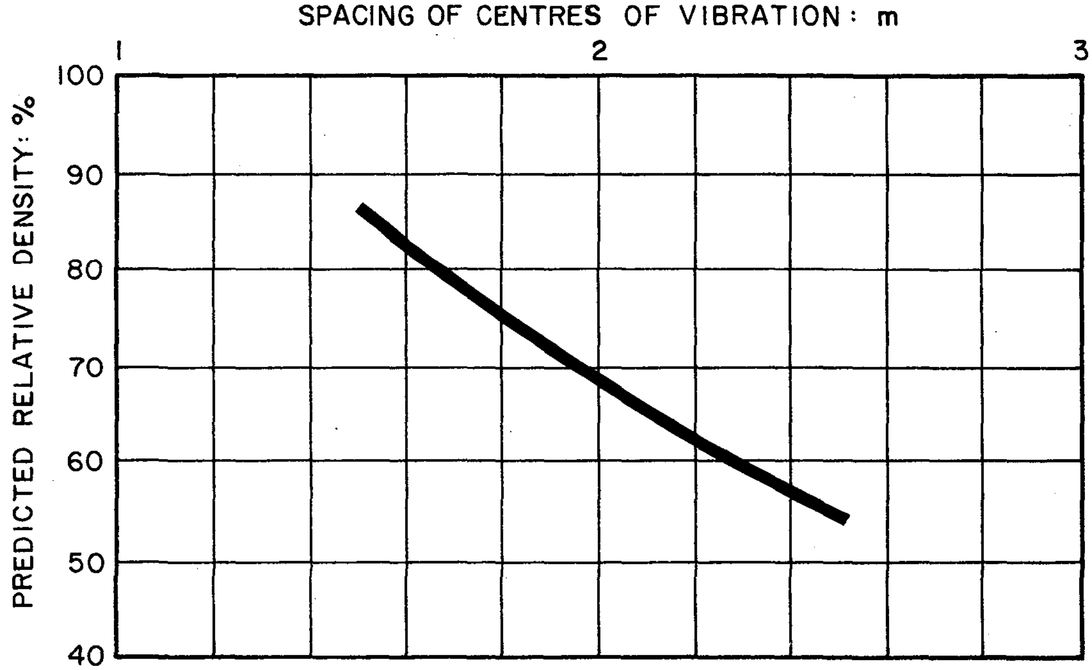


FIGURE 8 RELATIONSHIP BETWEEN RELATIVE DENSITY OF A CLEAN SAND AT POINTS MIDWAY BETWEEN THE CENTRES OF VIBRATION AND THE SPACINGS OF THE VIBRATION CENTRES (AFTER THORBURN, 1975)

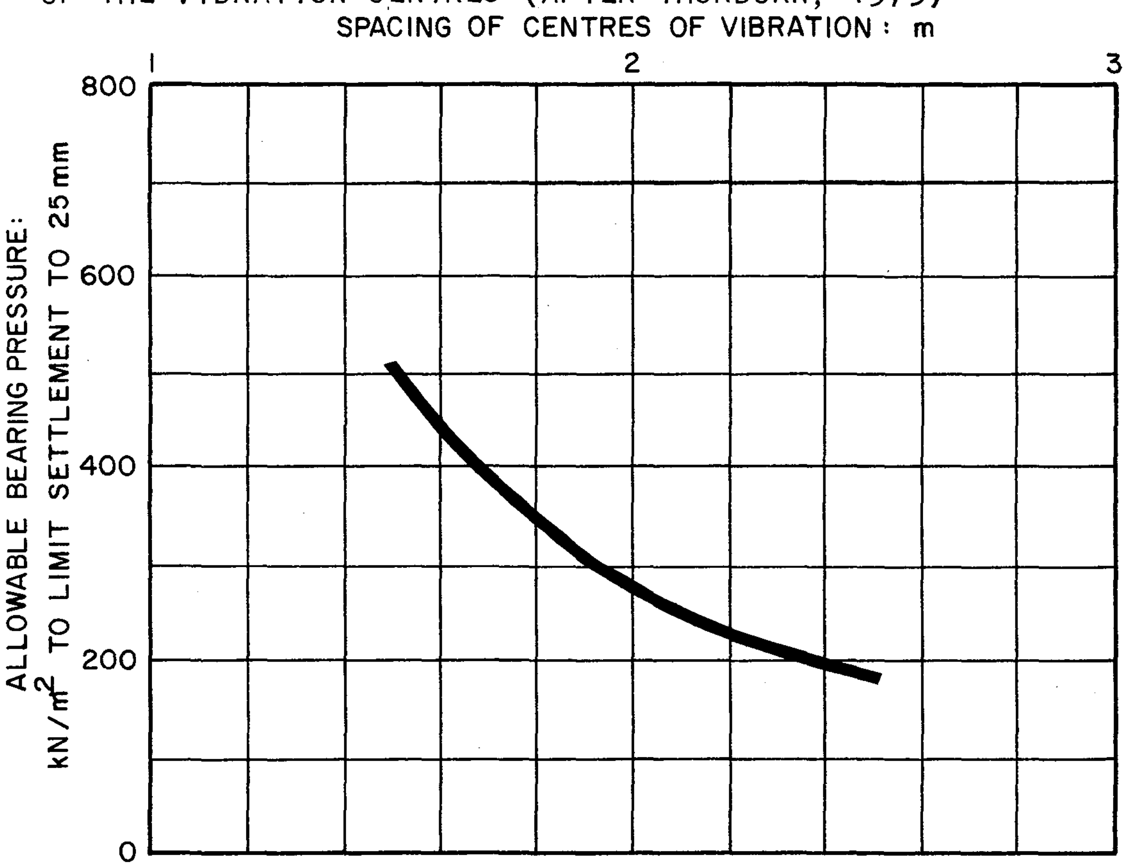


FIGURE 9 RELATIONSHIP BETWEEN ALLOWABLE BEARING PRESSURE AND SPACING OF VIBRATION CENTRES FOR FOOTINGS HAVING WIDTHS VARYING FROM ONE TO THREE METRES, FOUNDED ON COHESIONLESS SOIL (AFTER THORBURN, 1975)

more vibratory energy and thicker, heavier probe material. The length of the probe is 10 to 15 ft (3 to 4.6 m) longer than the maximum penetration depth. This is to allow for any flexing of the probe, particularly when probes over 50 to 55 ft (15 to 17 m) are used. This also allows for any cut-off requirements during the application.

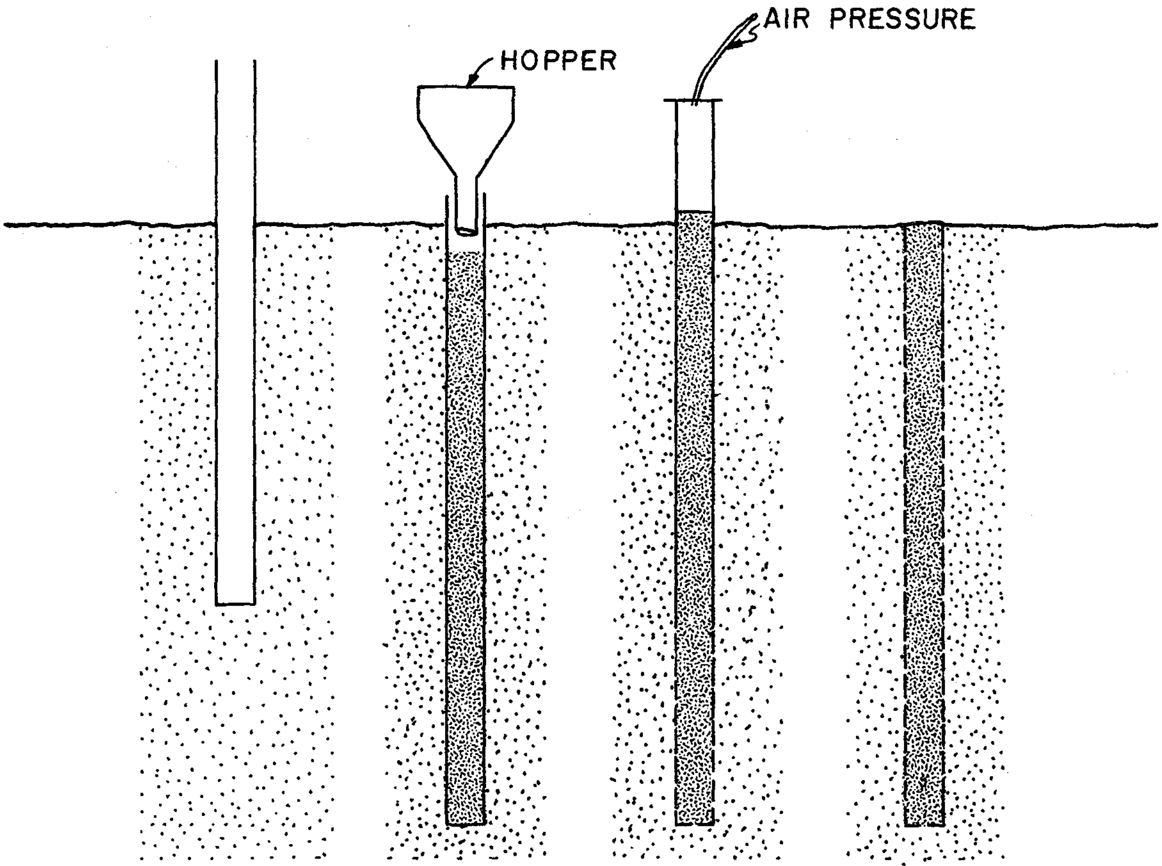
The types of soil that are suitable for the Terra-Probe application are believed to be sand similar to those applicable to the vibroflotation applications. To date, with this method 3- to 8-foot (0.91 to 2.44m) spacings have generally been used. An overburden of sand is required at the outset to allow for shrinkage caused by the compaction process, since material generally is not added during the application. About 12 percent shrinkage allowance has been sufficient for most applications, but a hydraulic fill hydraulically placed with its very low density will require about a 15 percent allowance.

In a recent comparative study (Brown and Glenn, 1976), the Terra-Probe and the vibroflotation methods were used to compact a clean, medium sand to about a 35-foot (10.7 m) depth at varied spacings of 8 to 12 ft (2.44 to 3.66 m). The Terra-Probe penetration-extraction rate was about four times greater than that of the vibroflotation method. However, the Terra-Probe method required about four to five times more probes than vibroflotation to achieve equivalent density. The maximum densities achieved with vibroflotation were significantly greater than achieved with Terra-Probe regardless of probe spacing. Comparison by Audibert (1977), however, showed that under similar conditions the Terra-Probe and vibroflotation methods worked equally well with a slight cost advantage for the Terra-Probe.

2.2.4 Compaction Piles

The use of piles to densify loose sandy soils derives its benefits mainly from displacement of material equal to the pile volume, although some densification may result from the effects of vibration during pile driving. While compaction piles could be timber or concrete structural piles, sand piles are generally used when the sole purpose is densification. The sand piles are usually installed by driving a hollow steel mandrel with a false bottom to the required depth, filling the mandrel with sand, applying air pressure to the top of the sand column while withdrawing the mandrel. The sequence of installing a sand compaction pile is shown on Figure 10 (Basore and Boitano, 1969).

The amount of compaction caused by driving piles into cohesionless soils has been considered by Meyerhoff (1956, 1959, 1960) and Robinski and Morrison (1964). The average increase in density over a site that is obtained when compaction piles are driven can be computed if the volume of driven piles and the surface settlement are known.



STEP 1 DRIVING HOLLOW STEEL MANDREL

STEP 2 FILL MANDREL WITH SAND

STEP 3 WITH DRAW MANDREL SAND COLUMN

STEP 4 COMPLETED SAND PILE

FIGURE 10 INSTALLATION OF SAND COMPACTION PILES (AFTER BASORE AND BOITANO, 1969)

Compaction piles employing low plasticity cohesive soils have been used to compact loess soil that collapses when saturated with water. The process consists of (1) sinking holes to the required depth in the foundation area by driving a seamless steel tube, by blasting, or by using a cable (churn) drilling rig; and (2) filling the hole thus obtained with soil, layer by layer; each layer is compacted with a rammer to a required density at near the optimum compaction moisture. The process is described in detail by Abelev (1975). The installation of soil piles enables the loessial soils to be compacted to a depth of 80 ft, achieving a stronger and less compressible mass no longer susceptible to collapse when wetted.

Table 5 summarizes examples of experience with compaction piles (Mitchell, 1970). It may be seen that pile spacings are generally in the range of 3 to 5 ft (0.91 to 1.52 m) and that the results obtained have been good.

2.2.5 Compaction Grouting

Grouting is a process of injection of a fluid material into a soil or rock mass for the purpose of stopping or reducing water movement, or consolidating and strengthening the soil or rock. While conventional or "penetration" grouting involved filling pore spaces in soil, joints or fractures in rock with a very fluid grout, "compaction" grouting consists of injecting under high pressure a very viscous grout into the soil to displace, compact, or densify loose or weak soils. A discussion of the application of penetration grouting for cementing or strengthening of soil mass will be presented in the section on Soil Strengthening. This section breifly reviews the use of compaction grouting as a stabilization technique for densifying in-situ soils.

While the technology of penetration grouting is fairly widespread and well documented, literature on compaction grouting is very limited. Graf (1969), Brown and Warner (1973), and Herndon and Lenahan (1976) recently summarized the state-of-the-art of compaction grouting, which provided the basis for this review.

The grout most often used consists of silty sand combined with water and about 12 percent cement to form a very stiff, zero-slump mortar-like mixture (Figure 11). Normally, two to four sacks of portland cement per cubic yard of sand are used, resulting in a material with unconfined compressive strengths ranging from 100 to 1000 psi (70 to 700 ky/cm^2). The silty sand may be untreated local materials or blended mixture containing 30 to 70 percent particles passing the No. 200 sieve. The silt content is necessary to provide plasticity and cohesion to the grout mix.

The grout holes are generally predrilled and cased at least from the surface down to the top of the zone to be densified. To prevent grout leakage, the casing (usually 2-in. (51 mm) ID) is cemented in-place. The hole is then extended 5 to 8 ft (1.52 to

TABLE 5

EXPERIENCE WITH DENSIFICATION OF GRANULAR SOILS USING COMPACTION PILES
(After Mitchell, 1970)

PROJECT	SOIL TYPE TREATED	PILE LENGTH	PILE TYPE	PILE SPACING	RESULT	COMMENT	REFERENCE
	Fine sand below water table	45 ft.	Cast-in-place concrete	3 ft.	Surface settlement = 3 ft. Pile volume equivalent to 1 ft. Porosity reduced from 44% to 38%.		Terzaghi and Peck (1967)
Foundations for Compressor Station	Coarse to fine sand	40 ft. 30 ft.	20-in. dia. sand piles	4x4 ft. 4x5 ft.	6-in. settlement due to pile driving. Sand pile volume equivalent to 30 in. layer. Void ratio decrease approx. 0.15. Settlements after construction of about 1/4 in.	Experience during project permitted increase in spacing and decrease in length as indicated	Swiger (1968) Personal Communication
General Experience	Loess	15 m.	Earth, 0.45 m. max. dia.	3.4 ft.	Can reduce porosity to <40%. Safe against collapse.	Earth rammed into drilled out casing	Lehr (1967)
Foundation Stabilization	Loess	12 m.	40-50 cm.	1 m.	Total settlement of 7.5 cm (rolling mill). Similar foundations on unstabilized soil settled 42 cm. in 8 years.	Rigid friction piles unsuccessful Mixed-in-place stabilized soil piles very effective	Abelev and Askalanov (1957)
Foundation Support for Building	Loose sand with clay inclusions	30 ft.	14-in. dia. sand	4-5 ft.	Obtained specified Relative Density of 75% in most of the area.	Clay inclusions gave erratic results	Woodward-Clyde and Associates* Project Report

*Published by Basore and Boitano (1969).

1 ft = 0.305 m
1 in = 2.54 cm

FIGURE 11 STIFF PLASTIC CONSISTENCY GROUT EXTRUDING FROM GROUT HOSE (AFTER BROWN AND WARNER, 1973)

2.44 m) beyond the tip of the casing by drilling through the casing. After grouting the extended hole, the hole is further extended by drilling through the casing and the "first stage" grout to form a second stage of 5 to 8 ft (1.52 to 2.44 m). The procedure is continued until competent bearing material is reached. Holes in excess of 100 ft (30.5 m) have been completed using this procedure. Grout holes may be inclined, but in most applications they are not more than about 20^{o} off vertical.

Alternatively, casing may be installed to the full depth of treatment and grout is injected as the casing is withdrawn. Although this method is more economical, it is not as effective, as grout leakage often results around the casing. The grout is usually pumped through a 2-in. (51 mm) or 2½-in. (64 mm) hose, 25 to 100 ft (7.62 to 30.5 m) in length. The maximum pressure used on most projects is within 50 to 500 psi (3.5 to 35 ky/cm^2) at the point of injection. Pumping pressures up to 1000 psi (70 ky/cm^2) can be achieved with recently developed equipment. Injection rates usually vary between 0.2 and 1.8 cu ft (5663 to 50,968 cm^3) per minute.

Compaction grouting has been successfully used to compact a wide range of soils including man-made fills, sands, silts, unsaturated clays, and organic soils and peat. Ground water does not adversely affect injection or effectiveness. Saturated clays cannot be effectively compacted with compaction grout, and compaction grouting is not normally effective in the surface 2 to 4 ft (61 to 122 cm) unless superimposed loads are present.

The present-day technique of compaction grouting is an outgrowth of the "mud-jacking" technique, which has been used fairly extensively for nearly 40 years to raise sagging pavement by injecting soil-cement grout under the pavement slab. Compaction grouting has subsequently developed into a more sophisticated technique, complex in procedure and relatively high in cost. In addition, it can only be used in situations where there is sufficient confinement to permit the needed injection pressures. Therefore, the use of compaction grouting is limited to zones of relatively limited volume and to certain special problems, such as underpinning and lifting foundations of structures.

The future application of the compaction grouting technique to stabilize a large area or stretch of highway subgrade, therefore, is rather limited. However, the technique may be useful for stabilizing the existing subgrade of a distressed pavement, as the drilling and grouting equipment may be placed away from the pavement area, thus resulting in minimum interruption to the traffic movement.

2.2.6 Blasting

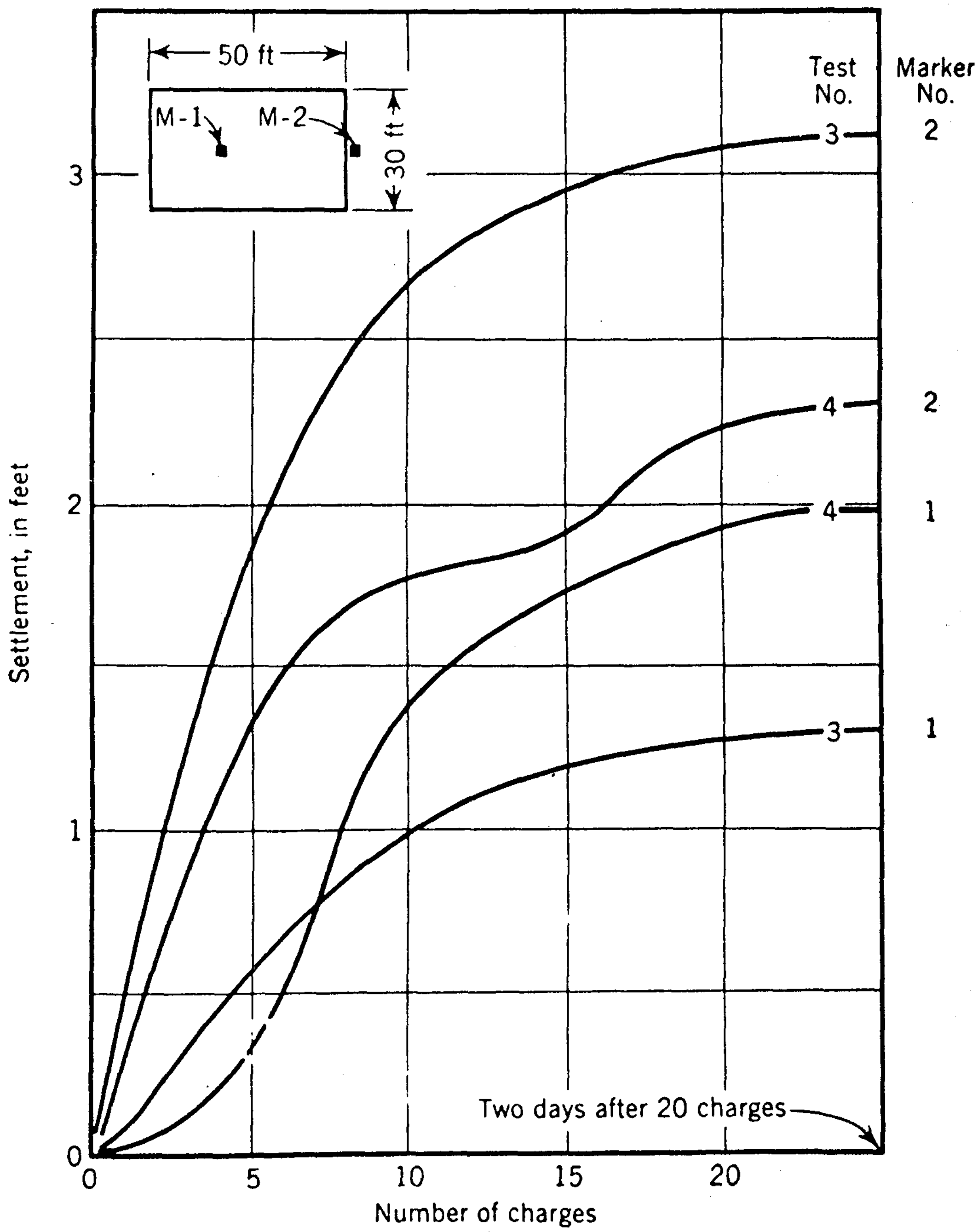
In the treatment of deep saturated, loose cohesionless soils by blasting, buried explosions are used to cause liquefaction followed by expulsion of pore water and densification. The general procedure for densification by blasting is to: (1) install a pipe to the desired depth (usually by jetting); (2) insert an explosive charge; (3) withdraw the pipe; (4) backfill the hole; and (5) fire the charges according to a preestablished pattern. While the escape of gas and water and the formation of sand boils at the surface are to be expected, actual cratering is to be avoided in the application of the method.

Mitchell (1970) presented an excellent summary of the applications of the blasting method for the treatment of deep, saturated, loose sand. The summary was later updated by Mitchell (1973). Readers are referred to these two references for more details.

Generally, densification can be obtained in soils of about the same range of grain size as can be densified by vibroflotation, although it has been used successfully in soils containing as much as 75 percent finer than 200 mesh. Available data indicate that an increase of about 15 to 30 relative density points can be obtained. In some cases, however, loosened zones up to several feet in diameter have formed around blast points. Figure 12 shows an example of ground settlement as a function of number of explosive charges, and illustrates that succeeding detonations become progressively less effective. Individual charges of from less than 1 to about 15 pounds (0.45 to 5.45 kg) at horizontal spacings of 10 to 25 ft (3.1 to 7.6 m) are typical. Best effectiveness is generally obtained when the center of charge is at a depth of about 2/3 the thickness of the stratum to be densified.

Table 6 presents examples of experience with densification of granular soils using explosives. The table was originally published by Mitchell (1970) and has been updated with a few recent examples.

With the technique of blasting using buried explosives, little densification results in the upper 3 ft (1 m) of sand. Therefore, the surface zones must be compacted separately by rollers (see section on Surface Compaction). There has been a study made, however, on the detonation of a blanket of foam-type of explosive for surface compaction of very loose sand and silt near 0 percent relative density (Goodman, Aidun and Grove, 1965). The foam propellant formulation contains hydrazone, ammonium perchlorate, a hydralized protein base foam liquid, and a foam stabilizer. The detonation of foam propellant on soil has shown a significant increase in cone penetrometer readings and soil density up to more than 15 inches (38 cm) in depth. Dry densities as high as 90 percent of the Standard AASHO maximum density and 100 percent of the Modified AASHO maximum density were obtained for a well-graded sand and for a clean bank run gravel, respectively. Increasing the amount of propellant per unit area yielded more



1 ft = 30.48 cm

FIGURE 12 GROUND SETTLEMENT AS A FUNCTION OF NUMBER OF EXPLOSIVE CHARGES (AFTER MITCHELL, 1973)

TABLE 6

EXPERIENCE WITH DENSIFICATION OF GRANULAR SOILS USING EXPLOSIVES
(Updated After Mitchell, 1970)

PROJECT	SOIL TYPE TREATED	THICKNESS OF LAYER FT.	TYPE OF EXPLOSIVE	CHARGE PATTERN	RESULTS	COMMENT	REFERENCE
Franklin Falls Dam Foundation	Loose, saturated fine sand	15-30	8 lb charges	5 ft. grid at 15 ft. depth	Surface settlement of 2 - 2-1/2 ft. Rel.density increased to 46-56%	Densified 400,000 yd^3 Average cost 9¢/yd^3	Lyman(1942)
Denison Dam Tests	Loose sand and silt with some clay	30	Dynamite 16 lb charges	16 ft. depth	Surface settlement of 0.5 to 0.7 ft.		Lyman(1942)
Karsafuli Dam Foundation	Uniform fine sand		3 and 8 lb charges	20 ft. spacing, depths of 15, 30, 50 ft. Lowest fired first, then middle and upper at 4 hr. intervals 4th series at 25 ft. depth	Max. surface settlement of 2.7 ft. Porosity decreased from 47 to 41%	Densified 60,000 yd^3 in a pothole	Hall(1962)
Tower Foundations	Loose, uniform fine to very fine micaceous sand with 7-75% silt	24	1/2 and 4 lb charges	8-4 lb charges fired simultaneously at 12 ft. depth, repeated three times; 24-1/2 lb charges at 6 ft. depth 12 ft. grid pattern	1-1/2 ft. maximum settlement. Tower settled max. of 0.11 ft. after construction (250 ton vertical force)		Wild and Haslam(1962)
Investigation of Sand Liquefaction Danger	Saturated sands	25-30	5 lb ammonite	4-5 m. depth, Three blasts in succession.		If average settlement <8-10 cm with radius of 5 m. then liquefaction danger small	Abelev and Askalanov(1957) Florin and Ivanov(1961)
Investigation of Stability of Marine Deposits Against Flow Slides	Sands and silts		Dynamite and plastics	0.07-2.4 kg charges at 5-10 m. depth			Kummeneje and Eide(1961)

1 lb = 0.454 kg
1 ft = 0.305 m
1 yd^3 = 0.764 m^3

TABLE 6 (continued)

PROJECT		SOIL TYPE TREATED	THICKNESS OF LAYER FT.	TYPE OF EXPLOSIVE	CHARGE PATTERN	RESULTS	COMMENT	REFERENCE
Verdalsora		Fine sand and silt		Dynamite	1 at 0.3 kg, 1 at 0.6 kg, then 5 in succession at 1.2 kg each	Surface settlement of 1.3 ft. Porosity reduced 3-5%. Rel. density increased from 44 to -60%	Zone of 6 m. dia. around blast point was loosened	Kummeneje and Eide (1961)
Trondheim Harbor		Fine sand and silt	50-65		0.07-1.4 kg at 30 ft. depth			Kummeneje and Eide (1961)
Vaernes		Loose fine sand		Dynamite	5 shots for total of 6 kg	1.1 ft. settlement at center of depression		Kummeneje and Eide (1961)
Five Projects in Florida	1-3	Fine medium uniform sand	7-18			Rel.Density increased from -40 to 80%		Prugh (1963)
	1-4	Fine medium uniform sand	7-18			Rel.Density increased from -50 to 71-96%		Prugh (1963)
	4	Fine silty sand	16			Rel.Density increased from -78 to 99%	Contained clayey sand pocket - no increase in Rel. Density	Prugh (1963)
	7A	Fine sand, silt shells	2-6			Rel.Density increased from -55 to 70-93%		Prugh (1963)
	7B	Fine uniform sand	10			Rel.Density increased from -65 to 90%	Surface settlement of 0.5 to 0.7 ft.	Prugh (1963)
Unknown		Damp fine sand	10-20			Rel.Density increased from 50 to 90%	Created vertical hole with 1st explosion, densified the saturated backfill with 2nd explosion	Prugh (1968)

TABLE 6 (continued)

PROJECT	SOIL TYPE TREATED	THICKNESS OF LAYER FT.	TYPE OF EXPLOSIVE	CHARGE PATTERN	RESULTS	COMMENT	REFERENCE
Unknown	Loess	60			Porosity reduced from 42-46 to 40%	Flooding through a series of wells followed by blasting	Litvinov (1966, 1969)
Hydraulic Fill Dam	Sand		High voltage electric charges	Triangular - 10 ft spacing	Surface settlement of 1-5 in.	Raising the electrode 10 ft. after 20-30 discharges	Civ. Eng. Pub. Works Review (1963)
Landslide	Sensitive, soft clay			Small, delayed charges 8-10 ft. spacing		To remold the clay by small charges and let the clay to reconsolidate	Lone and George(1967)
Unknown	Loess		Separate, chained cartridge			Created vertical hole with explosives, rammed soil pile in the hole	Abelev(1975)
Experimental	Very loose sand and silt		Foamed	Blanket of propellant	Achieving cone penetration resistance 10-40 psi	Surface compaction to a depth of more than 1 ft.	Goodman, et al. (1965)

effective results at deeper depths. Multiple detonation techniques promise to be the most effective. No practical case of application has been found in the literature.

2.3 STRENGTHENING

2.3.1 Vibro-Replacement

In recent years, the vibroflotation method has been extended to the construction of foundations in soft, cohesive soils and organic deposits. The vibroflot forces a vertical hole through the soft ground, which is then backfilled with gravel or crushed stone 1-inch (25 mm) to 3-inch (76 mm) nominal size and compacted under the action of the vibroflot. Densified columns of stone are typically 2½ to 3½ ft (76 to 107 cm) in diameter and are arranged in triangular grid patterns at spacings of 5 to 9 ft (1.5 to 2.7 m). A 2- to 3-foot-thick (50 to 76 cm) granular fill placed over the tops of the columns aids in both load distribution and drainage. It has been found that when properly applied, the vibro-replacement technique significantly reinforces soft ground and provides substantial increases in bearing capacity and decreases in settlement as compared to the untreated ground.

The equipment used for this type of construction is the same vibroflotation equipment used to densify base, cohesionless sand deposits. The stone columns have generally been used directly under isolated footings and large mat foundations; therefore, they are usually extended to firmer underlayers. The stone columns have also been used beneath embankments to increase the supporting capacity of the soft ground. The columns generally act initially as relatively flexible piles that dilate diametrically and develop lateral resistance in the surrounding soft material. At the same time, pore pressure generated by the loading is quickly dissipated by drainage through the columns so that the rate of consolidation of the soft ground is accelerated, resulting in rapid settlement and early strength improvement.

When the vibroflot penetrates without jetting water, it tends to displace the soil laterally, resulting in local compaction of soil around the boreholes or heave at the surface. The borehole is almost the same diameter as that of the vibroflot that must be withdrawn completely to facilitate backfilling. The suction developed thereby is conducive to collapse of very soft material, especially below ground water. Therefore, water circulation should always be used where the borehole is unstable or where natural ground water stands within the depth of the borehole. Greenwood (1970) stated that cohesive soils of strength less than 400 to 500 psf (1.95 to 2.44 T/m^2) cannot be treated by dry techniques. However, with the wet techniques, stone columns can almost always be formed in soft grounds of strength greater than approximately 150 psf (0.73 T/m^2). Circulation of compressed air through the vibroflot to generate a back pressure in the borehole improves stability, but below ground water level, the rising air bubbles increase the risk of collapsing the hole.

This method of replacing portions of soft cohesive soils was utilized for a gravity pier at Hunter's Point Navy Yard in San Francisco. In this particular case, sand-filled piers were constructed using the discharge of a hydraulic dredge to jet out holes that were from 4 to 8 ft (1.22 to 2.44 m) in diameter and extended for 10 to 20 ft (3.1 to 6.2 m) into soft bay mud. The gravity pier structure was then supported on this sand replacement type pier. Such a method has not had any broad application in highway construction, although intuitively it can be utilized in areas of very soft clays.

Under concentrated loading, such as beneath a small footing, failure of the stone columns usually occurs by bulging near the top where the major to minor principal stress ratio is maximum. Under widespread loads such as areal fills, the lateral restraint for the columns is much greater, and the total carrying capacity of the column is determined by local shear failure in the soil surrounding the column or by the end bearing capacity of the column.

Thorburn and MacVicar (1968), Hughes and Withers (1974), and Hughes, Withers and Greenwood (1975) studied methods for estimating the ultimate column load and settlement of the stone columns. Accurate estimation of column diameter is the major factor influencing the calculation of ultimate load and settlement characteristics.

Generally, the design bearing capacity of stone columns is in the range of 10 to 40 kips (50 to 150 KN) and settlement of the individual stone column may be expected to range from ½ to 1 inch (12 to 25 mm). Figure 13 shows the relationship between allowable working load on a stone column and undrained shear strength of cohesive soil at the depth of maximum radial strain of the stone column, as suggested by Thorburn (1975).

The above design parameters provide a measure of the capacity and performance of the individual stone columns. They only reflect the quality of the stone columns under confinement of the surrounding soils to withstand concentrated loading directly on the stone columns; they do not represent the overall quality of the reinforced ground. Therefore, under large areal loads such as beneath a roadway, embankment or a large mat foundation, the performance of the ground reinforced by the vibro-replacement technique can be expected to differ significantly from the performance of the individual stone columns. Watson and Prugh (1968) reported large-scale tests on a 25-foot-square (7.62 meter-square) concrete slab performed on ground treated by the vibro-replacement method. The ground was underlain by strata of soft silt, peat, and hydraulically placed pulverized fuel ash, varying from 15 to 30 ft (4.6 to 9.2 m) in thickness. A dry vibro-replacement process was used. After formation of a nearly cylindrical hole, granular fill (hard, inert 4-in. (102 mm) graded blast furnace slag) was placed in layers. Each layer was densified by vibration, and the resulting column diameters ranged from 24 to 48 inches (61 to 122 cm) depending on

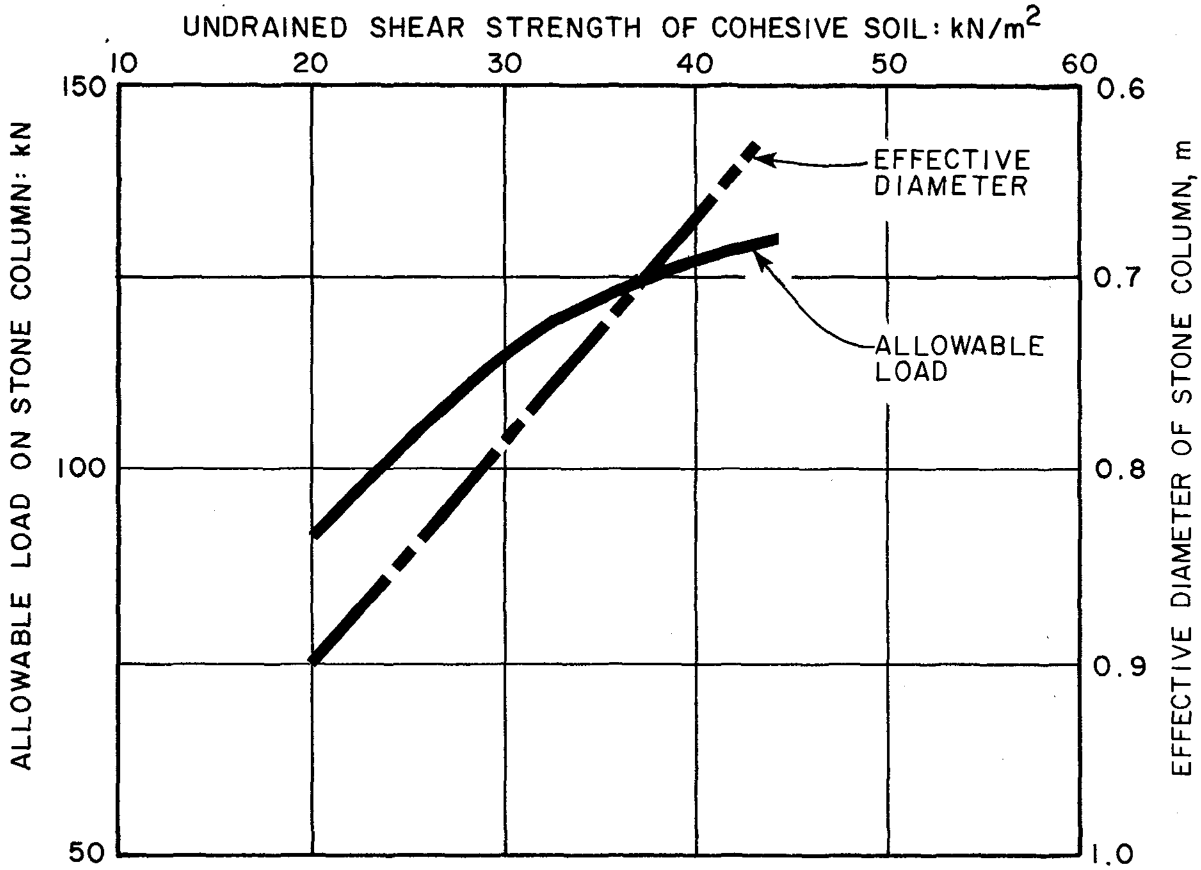


FIGURE 13 RELATIONSHIP BETWEEN ALLOWABLE WORKING LOAD ON STONE COLUMN AND UNDRAINED SHEAR STRENGTH OF COHESIVE SOIL AT POINT OF MAXIMUM RADIAL RESISTANCE (AFTER THORBURN, 1975)

the condition of the ground. Stone columns were spaced at 7.5 to 9.0 ft (2.3 to 2.7 m) in a triangular pattern. The large-scale load tests showed settlements of 1.25 and 4.4 inches (31.7 and 111.8 mm) under applied pressures of 0.5 and 1.0 tsf (4.9 and 9.8 T/m^2), respectively. For untreated ground, however, the settlement was measured to be 0.2 in. (25 mm) under a 1.0 tsf (9.8 T/m^2) load.

A 24-in.-diameter (73.2 cm) plate-bearing test was also performed on the treated ground midway between stone columns at 7.5-foot (2.3 m) centers and on the natural uncompacted fly ash fill material. Moduli of subgrade reaction derived from the plate-bearing tests were 140 and 20 pci (3875 and 553 gm/cm^3) for treated and untreated grounds, respectively, an improvement of approximately seven times.

McKenna, Eyre and Walstenholme (1975), however, reported an unsuccessful experiment with the vibro-replacement method beneath a high embankment. At the site, 50 ft (15.2 m) of loose to dense silty sand was overlain by about 40 ft (12.2 m) of soft silty clay with a peat layer. The stone columns were about 30 in. (91.5 cm) in diameter and 37 ft (11.3 m) long, to the top of the silty sand stratum; they were constructed on a triangular grid at 8-foot (2.44 m) centers. The embankment was built to a height of 26 ft (7.9 m), and the base of the embankment was about 230 ft (70.1 m) wide. The foundations of the embankment were instrumented, and the treatment ground showed no apparent effect on the amount and the rate of settlement of the embankment. A large section of the embankment failed at the height of 26 ft (7.9 m).

The ineffectiveness of the stone columns was attributed to two reasons. First, the stone column might have lost its effectiveness as a vertical drain to accelerate consolidation of soft clays and peat due to contamination of the stone backfill by clay slurry during construction. Secondly, the strength of the stone column might have been inadequate, which resulted in excessive lateral strains under the embankment loading.

Unconfined peat layers such as basin bog cannot be treated by vibro-replacement methods. Ground with peat layers not exceeding 2 to 3 ft (61 to 76 cm) in thickness and intercalated with sand or clay deposits can be treated effectively and economically with stone columns. Treatments of thick layers of confined peat and organic silty clays have been reported by Thorburn and MacVicar (1974) and Watson and Thorburn (1966).

Refuse fills, which may consist of sand, clay, brick, ash, rubble, and demolition debris, are usually very erratic in their compositional, as well as physical, characteristics. The state of compaction and their response to vibration are also more variable; however, they can be strengthened by the vibro-replacement method. Greenwood (1970) summarized the results of several cases of applications; this summary is given in Table 7. The test

TABLE 7

RESULTS OF *0.6 m DIA. PLATE BEARING TESTS ON FILLS*
(Greenwood, 1970)*

SITE	MEAN SETTLEMENT, *mm*	NUMBER OF TESTS	RANGE, *mm*	GROUND	REMARKS
Manchester Hulme V					
On cols	2.5	22	6.0	Clay, brick, rubble fill, demolition debris	1.5 m square compaction pattern beneath 3 m wide strips
Between cols.	3.7	3	1.2		
Rochdale					
On cols	1.8	3	0.5	Silty sand, ash, demolition debris, brick, rubble fill	2.1 m compaction spacing beneath narrow strip foundation
Between cols (wet technique)	4.5	2	1.1		
On cols	1.8	1	-		
Between cols (dry technique)	7.9	1	-		
Leeds, Balm Road					
On cols	11.0	2	16.0	Brick, rubble, ashes, sandy clay fill	Test load 305 kN/m^2, 2.1 m compaction spacing beneath narrow strip foundations
Between cols.	42.0	1	-		
Untreated	84.0	1	-		
Leeds, Belinda Street					
On cols	1.6	1	-		
Between cols.	4.6	1	-		
Untreated	12.0	1	-		
Birmingham, Hobmoor Road					
On cols	5.5	4	6.0	Clayey sand and gravel fill, refuse brick rubble	2.1 m compaction spacing beneath narrow strip foundations
Runcorn					
On cols	6.0	7	8.6	Uniform medium sand (old fill) uniformity coefficient, $D_{60}/D_{10}=2$	
Untreated	11.5	1	-		
Liverpool					
Between cols.	4.3	7	3.5	Brick, rubble, ashes, demolition debris, soil	Test load 310 kN/m^2

*Except where noted, a test load of 375 kN/m^2 was applied at 0.6 m below ground level. 1 kN/m^2 = 20 psf load shown in Table 7 is approximately twice the normal working load for housing sites; even under this overload, the total and differential settlements are well within normal building tolerances.

2.3.2 Chemical Injection

The injection of chemicals into a soil for the purpose of providing cementation to the soil particles and thus strengthening the total soil mass is synonymous with the technology of chemical grouting or penetration grouting. However, as discussed in the section on Densification, grouting is a complex and high-cost soil treatment method that can only be used in situations where the volume of application is limited and where there is sufficient confinement to permit the needed injection pressure. In addition, the results of grouting are difficult to examine and evaluate. Therefore, the use of chemical grouting as a general stabilization technique for use in pavement design and construction in difficult soils has not been widespread, and the future application of the technique in the area, in our opinion, also appears to be rather limited. A review of the vast body of knowledge that has been developed on chemical grouting would, therefore, not be consistent with the goal of this report. Readers are referred to excellent recent summaries by Mitchell (1970, 1973) and Herndon and Lenahan (1976) for the grouting technology.

This section will, however, present a recently developed chemical injection process, which has shown promise of being able to strengthen or stabilize soft clayey soils and expansive clays. The process is known as lime slurry pressure injection (Lundy and Greenfield, 1968; Ingles and Neil, 1970; Wright, 1973; Thompson and Robnett, 1976).

The effectiveness of treating highly plastic clays and expansive clay is widely recognized (Federal Highway Administration, 1973). Lime has been used extensively in pavement construction, in which lime varying from 4 to 10 percent by weight is mixed mechanically into the soil, and the lime-soil mixture is then compacted in thin lifts into a densely packed mass for use as a compacted subgrade, or base and subbase courses at shallow depths. With the relatively new lime injection method, however, a lime slurry is pumped into the soil under pressure, forming a network of interconnecting seams or channels as much as 20 ft (6.1 m) deep into the in-situ soil.

The stabilizing effects of lime have been attributed to the abilities of lime to modify the plasticity of the clays and to stabilize the swelling and shrinkage characteristics of expansive clays. Lime has also been known to react with the siliceous and aluminous clay minerals in the soil to form cementing gels that result in relatively rapid strengthening of clayey soils. The seams or channels formed during the lime injection process may also serve as a moisture barrier, preventing the movement of moisture either into or out of the soil, minimizing drastic moisture variation and thus eliminating significant shrinkage or swelling in expansive clays. In addition, the injection of lime slurry into the soil may be considered as a form of prewetting, which has been shown to be an effective method for controlling volume change in expansive clays.

Generally, lime slurry with about 30 to 40 percent concentration is injected with a nozzle pressure ranging from 50 to 200 psi (3.5 to 14.1 kg/cm^2) at about 3- to 5-foot (91.5 to 152.5 cm) injection spacings. A normal intake is about 10 gal/ft (125 l/m) of injected depth. For treating expansive soils, a treatment depth of 7 ft (2.13 m) is usually sufficient to be below the critical moisture change zone, although current equipment is capable of injecting to depths of 10 ft (3.1 m). Immediately following injection, lime slurry left on the surface from injection is mixed into the top 5 to 8 in. (127 to 203 mm) of the soil and is recompacted. It appears that clays respond to the lime injection treatment most effectively at times of maximum desiccation, when fissures in the soil facilitate the groutability or extensive distribution of the lime slurry.

Because of questions related to the groutability of fine-grained soils with a low solubility, particulate material such as lime at the time of injection, the insignificant rate of lime diffusion, and the generally slow rate of lime-soil reactions, conflicting results have been reported and the effectiveness of the lime slurry pressure injection method has been controversial (Graf, 1974; Thompson and Robnett, 1976). The results of field studies of Lundy and Greenfield (1968) and Ingles and Neil (1970) showed that injected lime slurry will penetrate only along fissures or planes of weakness but not pores, which apparently is the most favorable condition for achieving success of the pressure-injected lime treatment methods. Under appropriate conditions, which are not yet fully understood, the lime slurry pressure injection technique has shown to be relatively simple and economical, and has been successfully used in many projects such as underpinning of foundations, improving the stability of soft or expansive clays, and treatment of railroad and pavement subgrades.

An alternative form of lime treatment of relatively thick in-situ clayey soils is the use of drilled lime piles. Rural and Urban Roads (1963) reported the use of drilled lime piles in stabilizing highway subgrades in Oklahoma. The method consisted of augered holes 9 in. (228 mm) in diameter and 30 in. (76 cm) deep with 5-foot (152.5 cm) centers, putting 25 lbs (11.4 kg) of hydrated lime in each hole, adding water and backfilling.

Handy and Williams (1976) reported the use of the drill-lime technique to stabilize an active landslide in montmorillonitic clayey soils in Iowa. The process used was similar to the method of treatment for the Oklahoma highway subgrade except that quick lime rather than hydrated lime was used, and that the lime treatment was restricted to the apparent shear zone in the sliding mass. Movement of the slide ceased immediately after treatment, when soil strength increased due to a drying and expansion action of the quick lime. Chemical tests after 1 year indicate beneficial cementation of soil within 1 ft (30.5 cm) of a treatment hole. In-situ shear tests after almost 3 years indicated doubling of the soil cohesion a foot from the treatment hole and an appreciable increase in internal friction. The movement of ground water in the shear zone and the existence of extensive shear planes in the sliding mass might have aided the lime migration process.

Several other studies have considered the drill-lime procedure (Thompson and Robnett, 1976). In general, the results of this method of treatment have been erratic. It is apparent that the major factor limiting the effectiveness of the drill-lime procedure is the inability to achieve lime distribution throughout the soil mass.

2.3.3 Other Methods

2.3.3.1 Preconsolidation

Preconsolidation or precompression with or without the aid of vertical sand drains has been a widely accepted method for improving marginal land for the construction of buildings, airfield pavements, embankments, and a variety of tanks and waterfront structures. Large areas of poor soils, such as soft clays, swamps, and sometimes refuse fills, have been successfully stabilized. The precompression concept is relatively simple, but the technique for proper application of the concept is a rather complex and involved subject. Readers are referred to two state-of-the-art papers prepared by Johnson (1970a and 1970b) for further information.

2.3.3.2 Thermal Treatments

Both freezing and heating have been used for the in-place treatment of foundation soils. The techniques may be complex and the costs high. Artificial freezing is most often used as a last resort for temporary control of ground water in water-bearing sand stratum encountered during tunneling and shaft sinking. Stabilization by heating treatment has little, if any, application in the United States; however, it has been extensively used in eastern Europe and the USSR for altering the properties of dry or partially saturated weak clayey soils and loess, mostly for building foundations. Readers are referred to a brief summary by Mitchell (1970) for further information.

2.3.3.3 Chemical Admixtures

The use of additives, such as cement, lime, lime-fly ash, bitumen, and other organic and inorganic chemicals for the construction of pavement structures has been studied and used extensively in the past 40 years. The additives, when properly mixed in and compacted with in-place material, may improve the strength, volume change, compaction, waterproofing, and/or freeze-thaw characteristics of the soil for use as a pavement subgrade. Comprehensive reviews and bibliographies dealing with these stabilization techniques may be found in the Transportation Research Board Publications.

2.3.3.4 Electro-Osmosis

Electro-osmosis has been successfully used primarily for the purposes of dewatering and consolidating saturated clayey soils. In addition, electro-chemical effects and electro-chemical injection processes associated with the electro-osmosis application have resulted in strengthening of the clay soils. Readers are referred to Mitchell (1970) for an excellent brief summary of the fundamentals and applications of the electro-osmosis process.

Another type of stabilization, somewhat similar to electro-osmosis, involves the introduction of calcium carbornate or other solidifying salts by electrical methods. The procedure differs from electro-osmosis in that the positive and negative ions would be introduced through hollow electrolytic poles by means of aqueous solutions. Then, when positive and negative ions make contact in the soil, they form a sort of calcium carbonate or some other solidifying salt that can cause very soft materials to become quite stable. This type of work has been successfully accomplished in laboratory experiments. Some years ago, an engineer in Portland was trying to develop this method for commercial applications. The method has not yet been field tested, but rationally in special circumstances might be more effective than ordinary electro-osmosis.

2.4 PROTECTION AGAINST ENVIRONMENTAL FACTORS

It has long been recognized by highway engineers that the supporting capacity of pavements, especially that of the subgrade, varies with the seasonal fluctuation of moisture content and temperature in the pavement components. The variation of moisture content in the subgrade may be caused by seasonal fluctuation of the ground water table and difference in the rate of infiltration or evaporation through the pavement surface and shoulders at different times during the year. Moisture movement may also be caused by temperature gradients in the pavement and subgrade. Normally, temperature-induced moisture migration is of relatively low magnitude; however, the action of frost can cause an almost unlimited quantity of ground water to migrate into the freezing zone and become frozen. The resulting effects of volume change and subsequent thawing can be detrimental to the stability of the pavement structure.

Increases in subgrade moisture during the winter are a primary reason for the increased deflections and reduced pavement supporting capacity in the spring (Linell and Haley, 1952; Scrivner and Moore, 1968; Ring, 1974). The results of the 1958-1960 AASHO Road Test (American Association of State Highway Officials, 1962) showed that for a silty clay subgrade, a 2 percent increase in the subgrade moisture can result in a reduced strength of from 50 to 100 percent, measured in California Bearing Ratio (CBR) values. Figure 14 shows a typical plot of seasonal subgrade strength as measured by plate-bearing tests for different soils.

The effects of moisture variation in a subgrade consisting of expansive clays were studied in detail in Oklahoma (Haliburton, 1971). Generally, a 1 percent increase in the subgrade moisture caused a 0.1 in. (.254 cm) increase in the vertical movement of the pavement on Oklahoma highways, and the results of lateral expansion of the subgrade due to the moisture increase were shown to have even more profound effects on the pavement performance.

Research has also shown that stress-strain and strength characteristics of soils are affected by temperature (Highway Research Board, 1969), although the practical significance of such an influence is not fully understood. Studies made at the Bureau of Public Roads, now the Federal Highway Administration (FHWA), Test Track in Virginia show that expansion of air in the air-water system of partially saturated pavements undergoing daily warming cycles can generate excess pore pressures that reduce the shear strength of granular base courses (Barber and Steffens, 1958).

Even though the combined effects of the cyclic seasonal variations of moisture, temperature, and frost action, if any, for a given full cycle may be small, the cumulative effects of the environmental changes after several seasonal cycles are some of the most important factors, besides the repetitious effects of traffic loading, that cause progress deterioration of the pavement strength and that the design of a pavement must take into consideration. Various techniques have been used in the past to guard the pavement components against the effects of the environmental factors such as moisture, temperature, and frost. One group of techniques seeks to provide good soil properties for the design. Such techniques include:

1. Replacing moisture-sensitive and frost susceptible soils; and

2. Using a chemical admixture to render the soil insensitive to moisture variation and nonsusceptive to frost action.

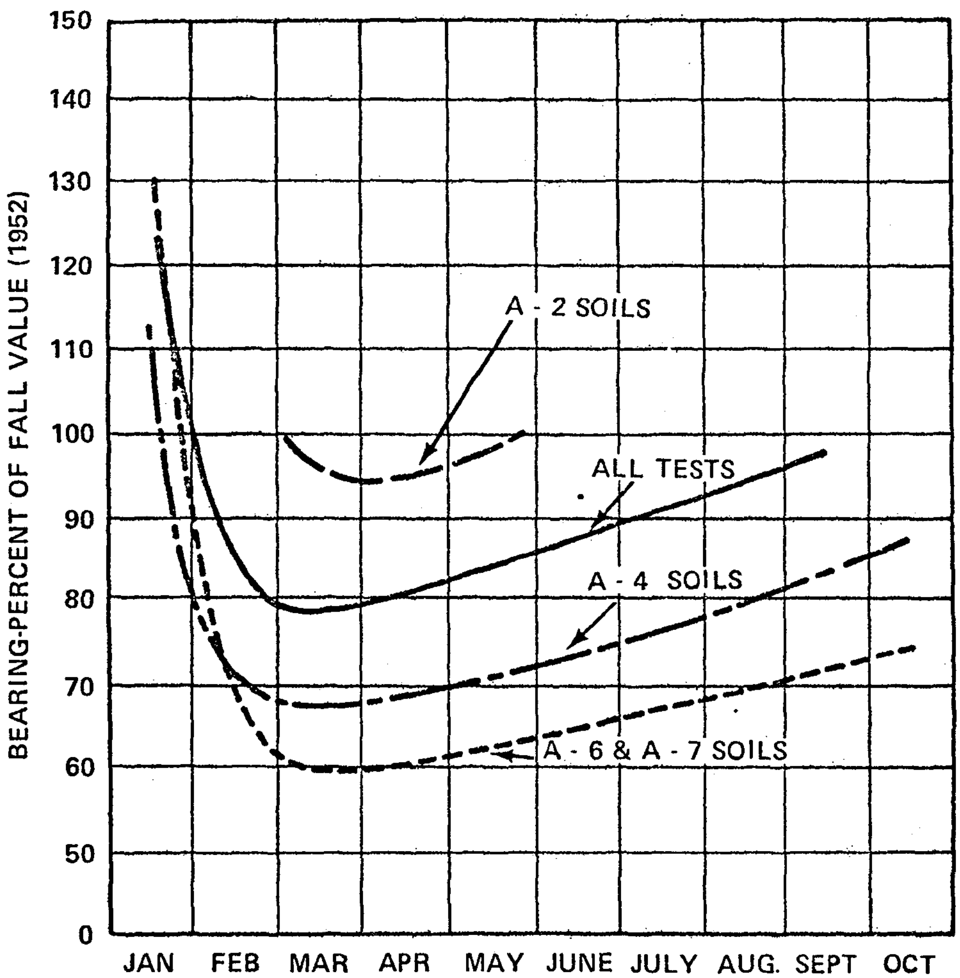


FIGURE 14 SEASONAL CHANGES IN BEARING CAPACITY AS MEASURED BY PLATE BEARING TESTS IN THE NORTHERN STATES (AFTER HIGHWAY RESEARCH BOARD, 1955)

The other group of techniques involves the process of protecting or preserving the existing soil properties as used in the design through:

1. Providing extra pavement component or thickness;

2. Protecting from moisture variation by encapsulation of the pavement components;

3. Insulating the pavement components from temperature variations or frost action; and

4. Maintaining a constant moisture condition in the pavement components by the use of drainage and/or cut-off soil layers or membrane.

The following subsections will present our review of the techniques of membrane encapsulation, insulation, and drainage.

2.4.1 Membrane Encapsulation

Most fine-grained soils have sufficiently high strength at low moisture content to satisfactorily serve as highway subgrade and subbases. However, an increase in water content, which may be accompanied by volume changes from either frost action on the expansive nature of the soils, will cause a significant loss in the strength of the soils. As a result, only a relatively small fraction of the strength that initially exists in the soil is used in the current design practice in order to take into consideration the saturated or near-saturated conditions of the soil after construction. If the soil that is sensitive to moisture variations can be isolated or protected from sources of migrating moisture, the design strength of the soil may be greatly increased, resulting in substantial savings in the costs of construction. Furthermore, the maintenance of a constant moisture condition in the pavement components preserves the permanency of the properties utilized for the design, which should prolong the life of the pavement structure and reduce the requirements for maintenance.

The use of an asphalt membrane as a moisture barrier in pavement construction dates back to as early as 1930 (Benson, 1953; Harris, 1963). Bell and Yoder (1957) advanced the concept of enveloping pavement components with a plastic membrane. Detailed studies were made regarding the permeability, strength, cost, and construction techniques on the use of vinyl and polyethylene sheets as moisture barriers to protect pavement subgrade and subbase from changes in moisture content. In recent years, the U.S. Army Corps of Engineers has experimented with the use of polypropylene, polypropylene-asphalt combinations and other plastic materials as moisture barriers. Terminologies such as Membrane-Enveloped Soil Layer (MESL) and Membrane-Encapsulated Pavement Section (MEPS) have since appeared in the literature (Sale, Parker and Barker, 1973; Webster, 1974). This subsection presents a review on the use of plastic moisture barriers.

There are basically two types of plastic moisture barriers. The first and simplest type functions as a barrier to cut off movements of water in the vapor phase, through capillary use from the ground water table, or from infiltration of surface water at the exposed surface. The cut-off barrier usually extends across the roadway section. The second type of barrier is a complete envelope around a pavement component that is sensitive to the moisture variations. The complete envelope is necessary when the cut-off barriers are absent or insufficient to prevent surface or ground water from moving into the pavement components. The two types of barriers are illustrated on Figure 15.

Bell and Yoder (1957) studied the effectiveness of plastics as moisture barriers and concluded that for a 6-in.-thick (152 mm) enveloped soil layer, the maximum increase in water content of the enveloped soils would be less than 1 percent in 10 years for a 4 mil vinyl plastic sheet and 1 percent in 100 years for a 4 mil polyethylene envelope. However, redistribution of water within the enveloped soil layer under severe frost conditions could cause an increase by 2 to 3 percent of water in the upper part of the soil layer. This increase in moisture content would be undesirable as it would bring about a loss of strength in the upper portion of the enveloped soil layer where stresses from traffic loads would be greatest. This moisture redistribution within the envelope may be minimized by initially compacting the enveloped soil at less than about 80 percent saturation. It appears that insulation of the pavement component from freezing temperatures in this instance, may also be advantageous (see the following section on Insulation).

Polyethylene sheets of 6 mil thick most often have been used as the cut off or the lower encapsulating membrane. Polyethylene is available from many sources in sheets of 100 ft long and up to 40 ft (12.2 m) wide. A typical 32- by 100-foot (9.75 by 30.5 m) sheet is supplied in a roll on an 8-foot-long (2.4 m) cardboard core and weighs 96 lb (43.5 kg) (Webster, 1974). Vinyl films are superior to polyethylene films with respect to puncture resistance. A vinyl film for the top membrane of an envelope must be at least 8 mils thick (Bell and Yoder, 1957).

The polypropylene fabric is a proprietary one and has been used successfully in conjunction with emulsified asphalt in forming the upper encapsulating membrane. Polypropylene is a black, nonwoven material, produced from petroleum-based products. It is generally available in 15-foot-wide (4.6 m), 100-foot-long (30.5 m) rolls, weighing 140 lb (63.5 kg) (Webster, 1974) and costing several times more than polyethylene.

Emulsified asphalt is usually sprayed under the polyethylene to serve as an anchor and also to seal punctures that may occur in the polyethylene during construction. It is also used to saturate the polyethylene fabric when forming the upper encapsulating membrane. The polypropylene has an asphalt retention of 270 percent (Sale et al., 1973).

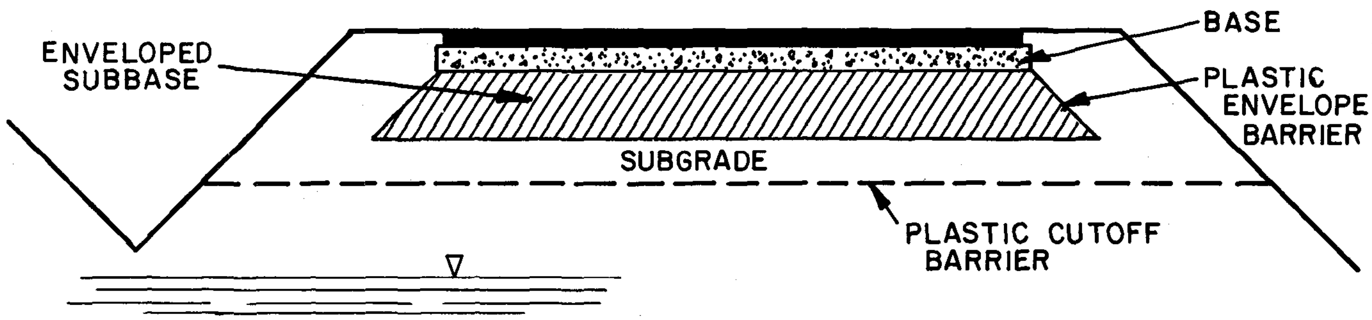


FIGURE 15 PAVEMENT CROSS-SECTION WITH PLASTIC MOISTURE BARRIERS (AFTER BELL AND YODER, 1957)

Typical construction procedures are described by Webster (1974). Special precautions are usually required to insure that coarse aggregates in the base or subbase will not punch holes in the encapsulating membrane. Bell and Yoder (1957) suggested that the compacted thickness of the soil layer above the plastic membrane be greater than twice the maximum aggregate size in order to minimize puncture.

Since fine-grained soils are usually most sensitive to moisture variations and frost action, soils used in the membrane encapsulated construction, therefore, include silty clays or clayey silts with more than half of the particles smaller than the No. 200 sieve size. The encapsulated soil must be compacted to a satisfactory density at a satisfactory moisture content, as evaluated by laboratory compaction and CBR data. Webster (1974) suggested that for base course, the encapsulated soil should be compacted to a minimum of 90 percent of the maximum dry density as determined by the AASHO T180 Method of Compaction at a compaction moisture at or slightly below the optimum moisture content; and that for subbase course, 95 percent of the AASHO T99 maximum dry density should be achieved at 2 percent below the optimum moisture content.

Conventional procedure of pavement thickness design is recommended for designing pavement containing membrane encapsulated soil sections. No consideration is given to the effects, if any, of the plastic membrane on the strength and deformation characteristics of the encapsulated soil layer. Based on limited field tests and available literature on these plastics, a service life of at least 20 years can be expected (Webster, 1974).

2.4.2 Insulation

The effects of cold temperature during the winter months on the supporting strength and deformation characteristics of the pavement components may be considered in two categories. The first is the distribution of existing moisture within the pavement structure due to the temperature differential between the upper portion and the lower portion of the pavement. Water tends to migrate in the form of vapor as well as in liquid through capillary action to the low-temperature zone in the upper portion of the pavement, resulting in weakening of the soil. The effects of redistribution of the moisture are usually not severe, especially when the existing moisture in the pavement is low and the moisture-sensitive, fine-grained soil is protected by other pavement components or insulated by artificial means.

The second category of effects of the cold temperature on the pavement components is frost action. Frost action occurs when a freezing temperature penetrates into the pavement components and causes water in the soil pore spaces to freeze. At the presence of frost-susceptible soils and internal sources of free water such as the ground water table, water would migrate from the ground water table through the capillary of the soil into the freezing zone and become frozen. When an excessive amount of ice lenses is formed, frost heave would occur. During spring thaw, drainage of the thawed water is usually prohibited by the frozen zone still present in the lower portion of the pavement. This supersaturated condition in the pavement components significantly weakens the supporting capacity of the pavement and causes numerous failures of the pavement during the spring.

Various methods have been used to design pavements in areas where severe frost is expected (Highway Research Board, 1963). All solutions to the problem of frost action are aimed at reducing or entirely removing one or more of the three factors contributing to frost action; the three factors are, as stated above, frost-susceptible soils, source of water, and freezing temperatures.

Generally, soils containing 3 percent or more of particles finer than 0.02 mm in diameter by weight are considered frost susceptible for pavement design purposes, according to the definition of the U.S. Army Corps of Engineers (Linell, Hennion and Lobacz, 1963). Many state highway departments, including those in Canada, consider soils with more than 15 to 25 percent of fines as being frost susceptible (Highway Research Board, 1965). Complete or partial replacement of frost-susceptible soils with clean, granular soils having low capillary conductivity to the depth of significant frost has often been used for pavement design and construction. Alternatively, chemical admixtures have been added to render the soils nonfrost susceptible. Extensive research and field experiments have been conducted in the area of chemical soil stabilization. However, the depth of treatment is usually rather limited and the permanency of the various treatment methods is not fully understood. A review of this vast area of information, therefore, is not conducted in this report.

Eliminating the source of water supply to the freezing zone can usually be accomplished through a lowering of the water table by subdrains and/or by installing cut-off barriers such as a clay blanket, clean granular layer, or plastic membrane. The concept of using a plastic membrane encapsulated pavement section was described in the previous section. A brief discussion on the use of subdrains will be presented in the next section on Drainage.

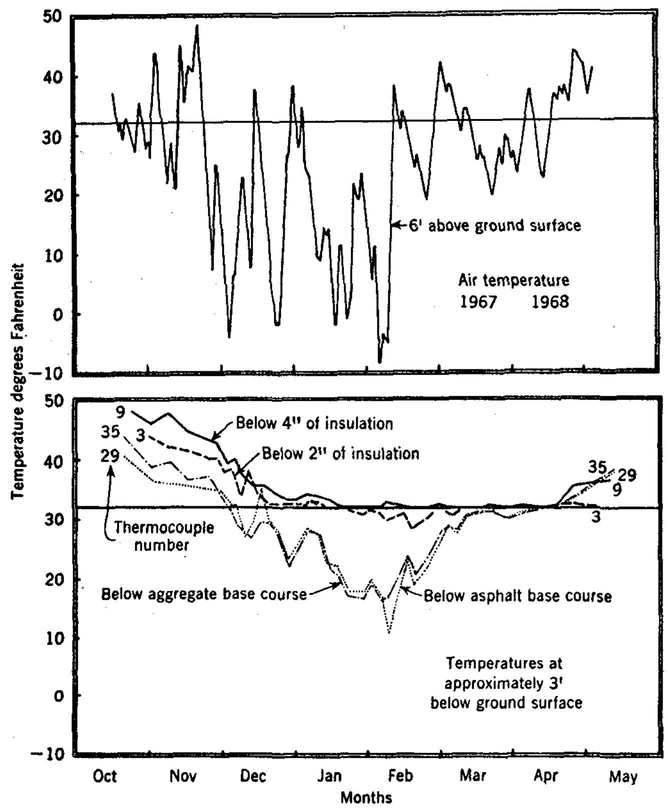
Reducing the penetration of freezing temperatures is usually achieved by providing thicker pavement sections over the frost-susceptible soils that comprise the subbase or subgrade. In recent years, many experiments have been conducted to insulate frost-susceptible soils with styrofoam plastics. This section presents a review of the styrofoam insulation technique based on literature published by Oosterbaan and Leonards (1965), Young (1965); Penner, Oosterbaan and Rodman (1966), Staub and Williams (1967), Keiner (1969), Becker (1972), and Robinsky and Bespflug (1973).

The styrofoam is a rigid polystyrene foam manufactured by Dow Chemical Company. One grade of the styrofoam used in the insulation application has a compressive strength of 35 psi at 5 percent deflection, and has a maximum water absorption of 0.25 percent of volume, a thermal conductivity of 0.23 BTU/hr-sq ft-deg F/in. (1851 J/hr-m^2-deg C/cm), and a density of 2.5 pcf (40.0 kg/m^3) (Keiner, 1969). The styrofoam is usually fabricated in boards 2 ft by 8 ft (61 by 244 cm), or 4 ft by 8 ft (122 by 244 cm) size in thicknesses of 1-1/2 (38 mm), 1-3/4 (44.5 mm), and 2 in. (51 mm). The sheets are placed on top of the frost-susceptible subgrade and held in place by wooden skewers. An asphalt emulsion may be applied for sealing joints between sheets. However, it was found that the cost involved in sealing and insulating a joint did not justify the small increase of thermal efficiency that would be realized (Oosterbaan and Leonards, 1965).

Figure 16 shows typical results of the effects of the use of styrofoam insulation to attenuate frost penetration (Keiner, 1969). Similar success has been noted in other experiments. It was noted that a 1-1/2 in. (38 mm) insulating layer effectively replaced more than 3 ft (91 cm) of granular material (Oosterbaan and Leonards, 1965). The foamed plastic insulating layer may, therefore, be effective in reducing the thickness of the flexible pavement structure normally required for satisfactory performance in a frost area.

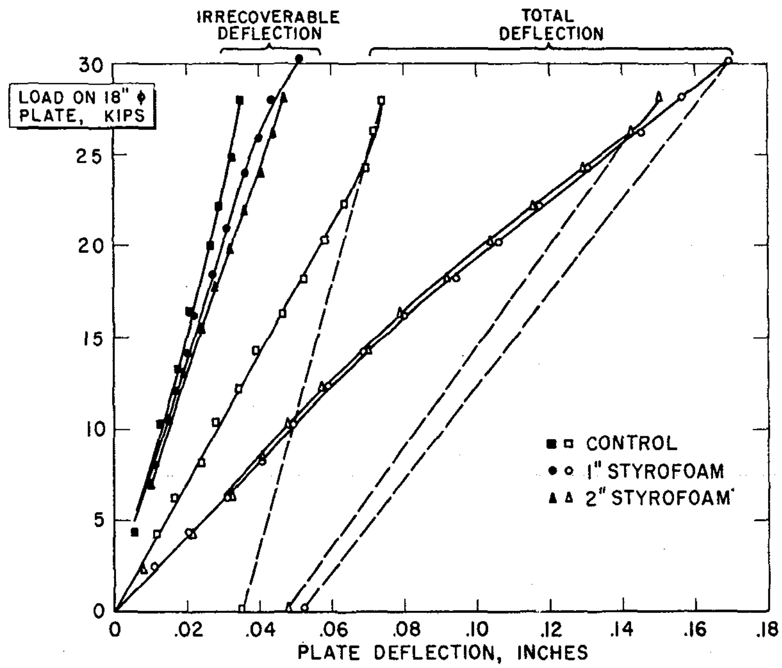
Results of plate load tests performed on test sections in Michigan showed that the insulating layer increased the elastic deformation of the pavement (Figure 17). However, the magnitude of the deflection was believed to be tolerable (Oosterbaan and Leonards, 1965). It has also been noted that the water content in the subbase over the insulation layer increased substantially during winter months because of the impremeable nature of the styrofoam material. In addition, the insulating layer may also cut off the upward conduction of heat from the warmer soil beneath the pavement, causing more frequent icing of pavements over insulated subgrade and affecting traffic safety (Ring, 1974; Withers, 1974).

The use of styrofoam insulation boards requires that the subgrade be fine graded so that deviations from a 10-foot (3.1 m) straightedge are in the range of $\pm$ 1/2 in. ($\pm$ 13 mm), in order to avoid puncture of the insulating board as a result of uneven subgrade when compacting the overlying layers. The insulating layer should extend across the pavement width to protect the edge of the pavement from heat loss through the shoulders. No special training is required for laborers to obtain a desirable quality of workmanship.



1 in = 25.4 mm
1 ft = 0.305 m

FIGURE 16 RESULTS OF THE USE OF STYROFOAM INSULATION TO ATTENUATE FROST PENETRATION IN AIRPORT PAVEMENT IN ALASKA (AFTER KEINER, 1969)



1 kip = 453.6 ky
1 in = 25.4 mm

FIGURE 17 RESULTS OF PLATE LOADING TESTS ON TEST SECTIONS, SPRING 1964 (AFTER 2 WINTERS OF SERVICE), MIDLAND, MICH. (AFTER OOSTERBAAN AND LEONARDS, 1965)

2.4.3 Drainage

The importance of providing adequate drainage in the pavement components has been explained in the previous sections. Subdrains are routinely provided in the subgrade to lower the ground water level beneath the pavement (Figure 18), or to intercept seepage from water-bearing stratum (Figure 19). Often base drainage is provided, as shown on Figure 20, in order to carry away water accumulated from surface infiltration. The material used for the base course should, therefore, be free-draining containing little or no fines, thereby precluding the possibility of subgrade saturation and pavement breakup due to freezing and thawing.

In cold regions, where frost action is severe, subdrains should be installed in accordance with details as shown on Figure 21 in order to maintain maximum effectiveness of the subdrains and minimize differential frost heave in the subgrade. Ice blockage of shallow drains may be alleviated through the use of pipes made of high heat conducting material and surrounded by a good insulator, as shown on Figure 22.

In recent years, the use of layered drains for roadbed drainage has gradually gained wide acceptance in highway construction (Cedergren and Lovering, 1968; Smith, Forsyth and Gray, 1970; Cedergren, Arman and O'Brien, 1972). The layered drains consist of a drainage layer composed of free-draining (i.e., open-graded) aggregates and a filter layer graded so that intrusion of the in-place soil into the drainage layer is prevented (Figure 23). The criteria for selecting a filter material for the prevention of erosion and the protection of drainage layers and perforated drain pipes are well known (U.S. Army Corps of Engineers, 1941); typical gradations and permeabilities of open-graded base and filter materials are shown on Figure 24.

Due to the difficulties and costs of obtaining quality filter materials in many locations of the country as well as the care required during construction, both woven and nonwoven plastic fabrics under different brand names have been increasingly used to replace granular materials as filters. Healy and Long (1971) experimented with the use of the filter fabrics around prefabricated, thin and channelized cores as subsurface drains. The results were termed effective for a wide range of soil types and were economically competitive with conventional mineral aggregate filter-drain systems. Dallaire (1977) recently summarized the use of filter fabrics; it appears that, despite its wide use and apparent success, the effectiveness of using filter fabrics directly on silt or clay soils is still questionable.

2.5 SUMMARY

As was stated in the Introduction of this Chapter, an ideal subgrade in the zero-maintenance pavement system must provide adequate supporting capacity, and be able to maintain the permanency of its design properties and carry out its design function

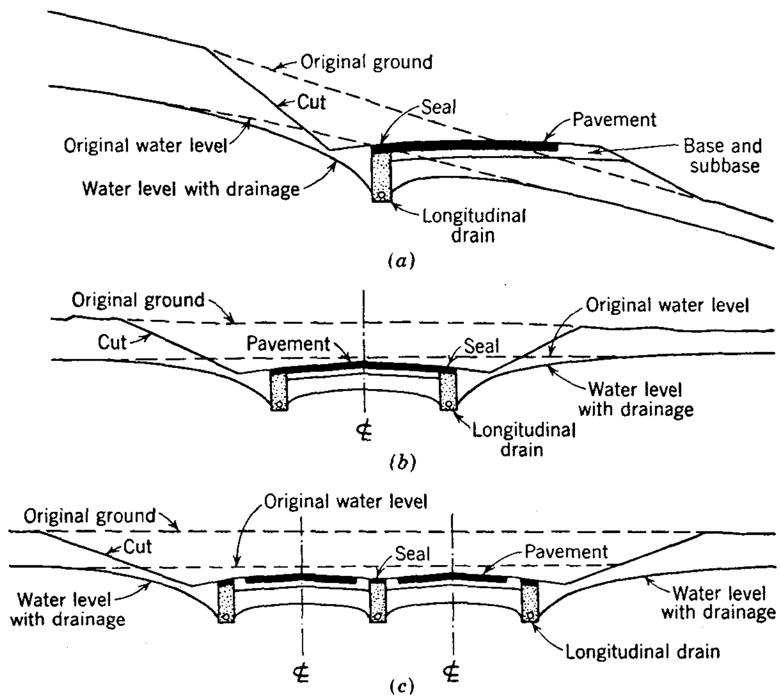
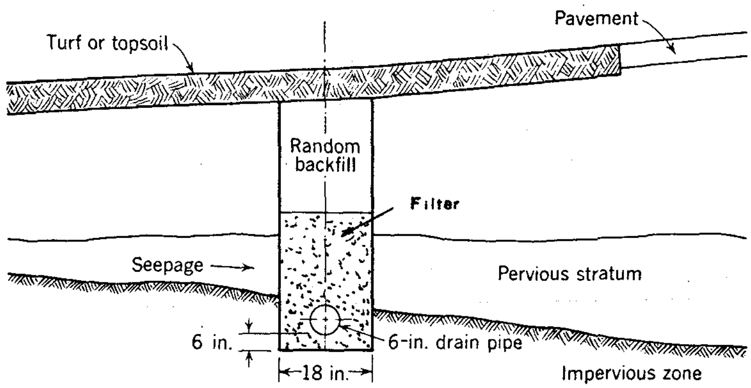
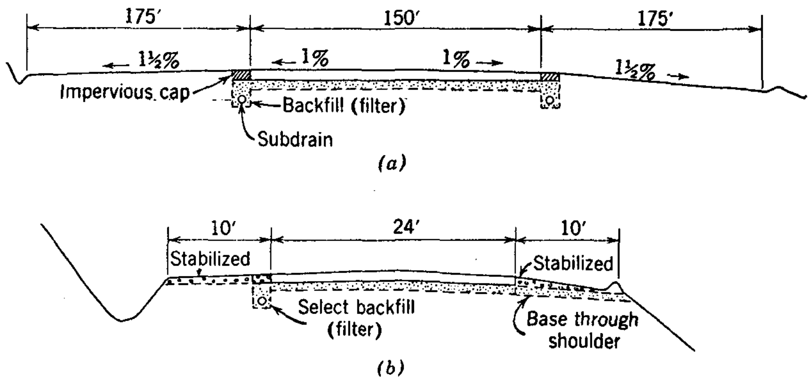


FIGURE 18 LONGITUDINAL HIGHWAY DRAINS (a) SIDE HILL CONSTRUCTION; (b) NARROW ROAD IN FLAT TERRIAN; (c) WIDE ROAD IN FLAT TERRIAN (AFTER CEDERGREN, 1967)



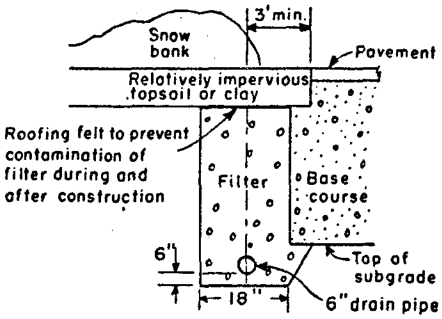
1 in = 25.4 mm

FIGURE 19 TYPICAL INSTALLATION OF INTERCEPTING DRAINS (U.S. ARMY CORPS OF ENGINEERS, 1955)

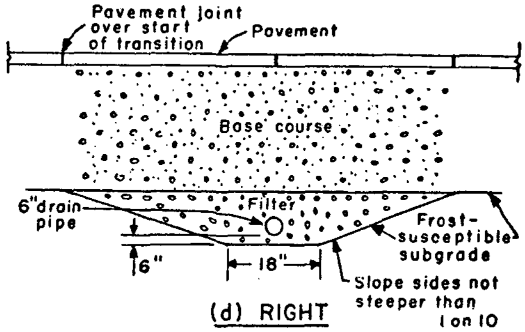


1 ft = 0.305 m

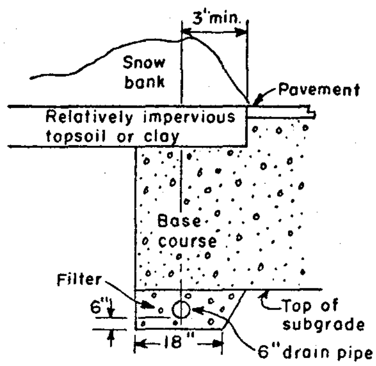
FIGURE 20 TYPICAL BASE-DRAINAGE INSTALLATIONS (a) AIRPORT BASE DRAINAGE; (b) HIGHWAY BASE DRAINAGE (NOT TO SCALE) (AFTER YODER AND WITCZAK, 1975)



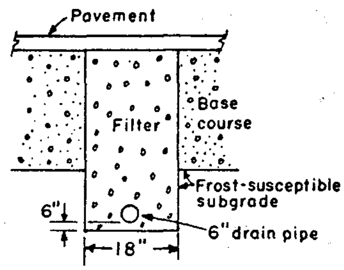
(a) RIGHT



(d) RIGHT



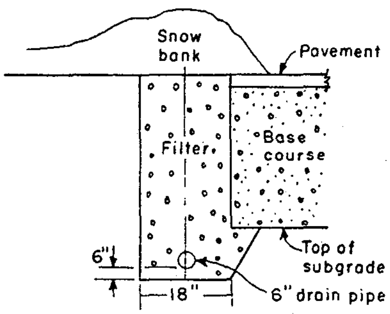
(b) RIGHT



(e) WRONG

Under frost conditions, non-uniformity produced by detail (e) is likely to result in differential heave and pavement cracking.

SUBDRAINS UNDER PAVED SURFACES



(c) WRONG

Under winter conditions, thaw water accumulating at edge of pavement may feed into base course in detail (c). This detail is poor because filter provides poor surface and is subject to clogging; drain is also located too close to pavement to permit easy repair.

SUBDRAINS ALONG PAVEMENT EDGES

NOTE:

For additional details on design of subdrains and filter courses see EM 1110-345-282.

1 in = 25.4 mm
1 ft = 0.305 m

FIGURE 21 SUBDRAIN DETAILS FOR COLD REGIONS (AFTER LINELL, HENNION AND LOBACZ, 1963)

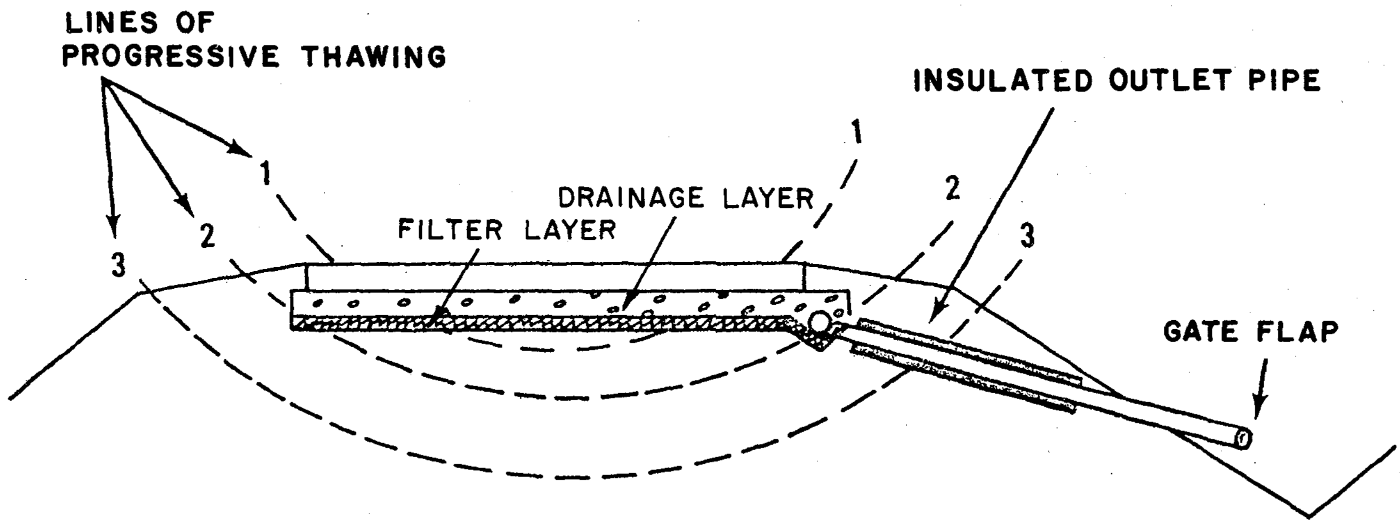
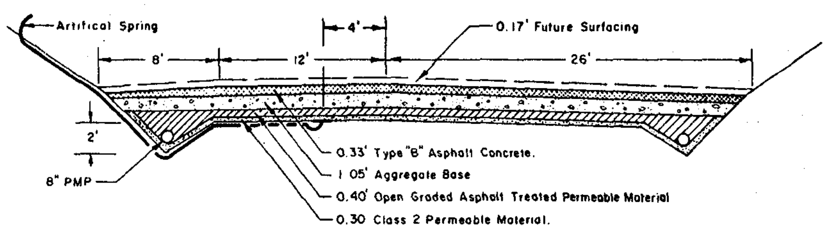


FIGURE 22 POSSIBLE SOLUTION TO ICE BLOCKAGE OF SHALLOW DRAINS (AFTER RING, 1974)



1 ft = 30.48 cm

FIGURE 23 GRADED LAYER DRAINS (AFTER SMITH, FORSYTH, AND GRAY, 1970)

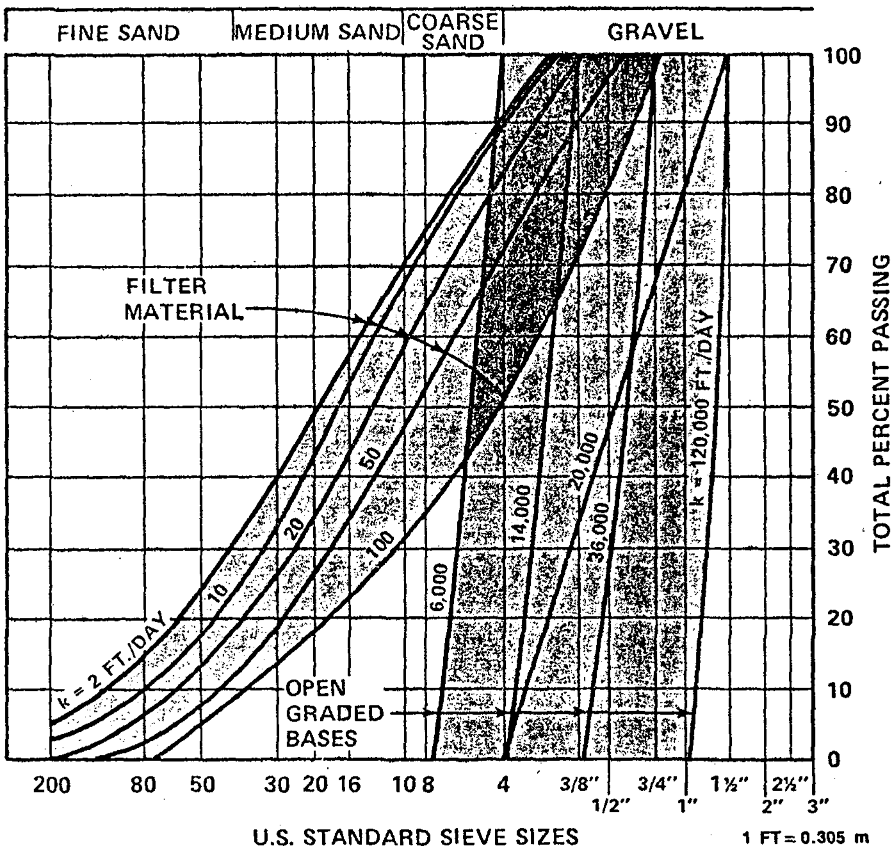


FIGURE 24 TYPICAL GRADATIONS AND PERMEABILITIES OF OPEN-GRADED BASES AND FILTER MATERIALS (AFTER CEDERGREN, ARMAN AND O'BRIEN, 1972)

with extremely small deflections and without requiring any maintenance during the entire design life of the permanent structure. This Chapter first presented the review of various special stabilization techniques related to densification and strengthening that may show promise in the ability to improve the engineering properties of in-situ soils for use as a subgrade for a zero-maintenance pavement. This Chapter then presented the review of techniques related to membrane encapsulation, insulation and drainage, which may be used to maintain or preserve the design properties in the subgrade during the design life of the pavement.

Conceptually speaking, the zero-maintenance pavement is achievable utilizing the conventional pavement system. The concept involves:

1. Selecting design parameters for the pavement components;

2. Using appropriate design procedures;

3. Enforcing strict quality control during construction; and

4. Maintaining or preserving the design parameters throughout the entire design life of the pavement.

Whether the conventional pavement system or a new, revolutionary pavement system is to be developed into a zero-maintenance pavement, it is likely that a sound subgrade or foundation that meets the requirements of an ideal subgrade will always be required.

Table 8 summarizes the major techniques reviewed in this Chapter. The suitable soil types that may be treated with each technique are indicated. The techniques related to membrane encapsulation, insulation, and drainage may also be applicable for treating base and subbase materials. Typical results of improvement, when available, and advantages and limitations of application for pavement design and construction are also summarized for comparison.

TABLE 8

SUMMARY OF SUBGRADE STABILIZATION TECHNIQUES

STABILIZATION METHOD	MAIN PURPOSE OF TREATMENT	SUITABLE SOIL TYPES	TYPICAL RESULTS OF IMPROVEMENT	ADVANTAGES	LIMITATIONS
Dynamic Consolidation	Densification	All soils incl. misc. fill and sat. clays	Ult. bearing capacity: 15 tsf for sand; Pressuremeter modulus: 1,000 psi for sand; Undefined shear strength: 600 psf for clay; to depth 30-100 ft.	Suitable for wide range of soils; feasible for large areas	Ground vibration; several weeks may be required between each pass of impact for cohesive soils
Vibroflotation and Terra-Probe	Densification	Sand less than 10% fine	To over 80% rel. density obtainable and modulus of subgrade reaction to over 600 pci; to depth 30 ft.	Feasible for treatment embankment and embankment foundation	For rel. small site
Compaction Pile	Densification	Fine to coarse sand, loess	To 75% rel. density in sand; 100 pcf dry density in loess at 3-5 ft. spacing; to depth 40 ft.	Use conventional equipment	For rel. small site
Compaction Grouting	Densification and reinforcing	Sands, silts, unsat. clays, organic soils, and misc. fills	Localized densification; raise distressed foundations or pavement; reinforce the foundation soils	Used normally for underpinning of foundations	Technique specialized and complex; used in confined and limited areas
Blasting	Densification	Fine sand and silts	Generally within 30 ft. depth; to over 70% rel. density and over 100 pcf dry density	Relatively economical	Ground vibrations; require surface compaction after blasting

1 psi = 0.0703 kg/cm^2
1 ft = 0.305 m
1 tsf = 9.765 T/m^2

TABLE 8 (continued)

STABILIZATION METHOD	MAIN PURPOSE OF TREATMENT	SUITABLE SOIL TYPES	TYPICAL RESULTS OF IMPROVEMENT	ADVANTAGES	LIMITATIONS
Vibro-Replacement (Stone Columns)	Reinforcing	Soft clays, misc. fills and organic deposits	Reduce settlement and increase bearing capacity	Feasible for treating embankment and embankment foundation	For rel. small site
Line Injection	Reducing shrinkage and swell; strengthening	Expansive clays and plastic clays	To depth 10 ft.; render expansive clays insensitive to moisture changes and slow strength gains	Relatively simple and economical; applicable to highway construction	
Membrane Encapsulation	Waterproof	Fine-grained soils	Cut off water migration and stabilize moisture in the pavement component	Successfully used in pavement construction	Extra labor in installation
Insulation	Frost protection	Frost susceptible soils	Reduce frost penetration significantly	Successfully used in pavement construction; easy to install	
Drainage	Lower ground water table and reduce moisture content	Beneath all pavement	Prevent saturation of pavement components from high ground water, surface infiltration, or frost action		

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CHAPTER 3

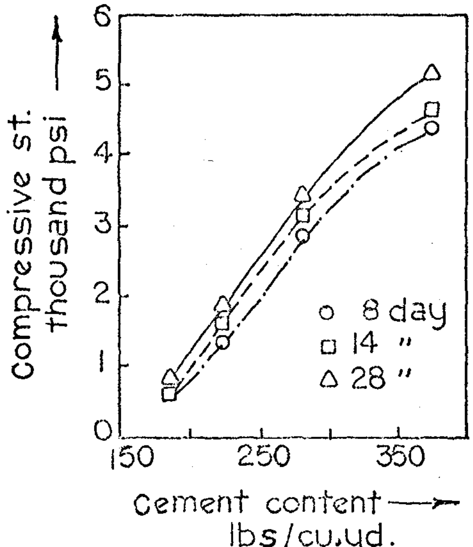
BASE AND SUBBASE

3.1 INTRODUCTION

This chapter presents innovative concepts suggested for use in the base and subbase. It is not the intent here to duplicate or reproduce data on chemically treated or cement and bitumen-stabilized base and subbase courses that are already in the domain of a conventional type of pavement material and being specified by many state and federal agencies. The conventional aggregate materials are on the verge of becoming exhausted, and there is a need to develop technology to use locally available aggregates and use them with admixtures (to improve properties) in the underlying layers. Therefore, a section of this chapter will discuss synthetic aggregates. New ideas of porous layers, fabric and other interlayers will also be discussed.

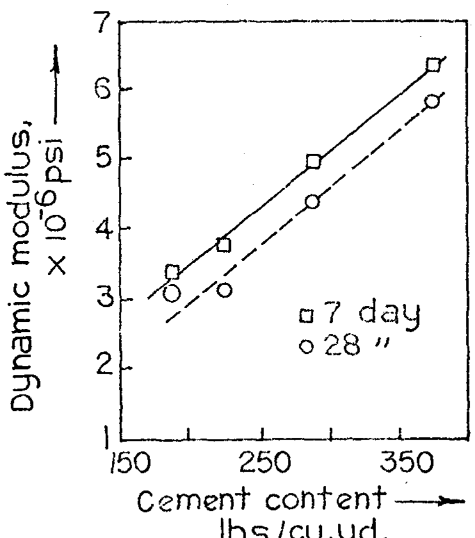
3.2 ECONOCRETE

W.A. Yrijanson (1977) describes econocrete as "a portland cement concrete designed for a specific application and environment and, in general making use of local commercially produced aggregates." Commercially produced aggregates refers to aggregates that are generally substandard compared to those usually specified in the conventional quality standard pavements. Aggregates that do not meet the standards of abrasion resistance, which become slippery under repeated loads, etc., can be used in the econocrete concept for the lower course in a composite pavement structure. For a zero-maintenance pavement, the surface course must be of a high quality, and with proper engineering considerations, it is possible to economize in the base. Currently the conventional pavements do not allow the use of some manufactured limestone or coarse aggregates that polish under traffic; gravels with an excessive amount of deterious material that cause pop-outs; high sand content mixes; sand shell aggregates; and aggregates with low abrasion resistance. The State of Florida has an abundance of low calcium vuggy oolite (a limerock) and the State of Iowa has many aggregate sources that will not be acceptable by current prevailing specifications. Table 3-1 is a typical gradation of a crushed limestone. Except for gradation (high fines content), this limestone meets the conventional quality aggregate. Figures 25 and 26 provide the strength and elastic modulus obtained from the aggregate gradation given in Table 9. The fines portion along with entrained air and water reducing admixture is a big help in providing a cohesive, workable mix with low cement factors. It should be noted that for any new material and gradation, the physical properties of concrete made from these materials must be determined beforehand, and only then can their proper design in a composite pavement system be undertaken.



(1 psi = 0.0703 kg/cm^2; 1 lb/cu yd = 0.593 kg/m^3)

FIGURE 25



(1 psi = 0.0703 kg/cm^2; 1 lb/cu yd = 0.593 kg/m^3)

FIGURE 26

TABLE 9

GRADATION OF CRUSHED LIMESTONE AGGREGATE
USED IN EXPERIMENTAL SET UP

SIEVE SIZE	% PASSING
1 in.	100.0
3/4 in.	95.2
1/2 in.	78.9
3/8 in.	69.6
No. 4	53.4
No. 8	37.0
No. 10	33.8
No. 16	25.4
No. 40	15.4
No. 50	13.6
No. 200	8.5

* 1 inch = 2.54 cm.

Very few composite pavements have been constructed in the United States; the ones that have been tried do present a future.

Tests have shown that a proper design mix - 200 to 250 pounds of cement per cubic yard (118.7 to 148.3 kg/m^3) - of econocrete can be used as a subbase. The low cement factor requires air entrainment (and water reducers sometimes) to obtain a plastic workable mix. Yrijanson (1975) remarks that an econocrete subbase would provide a very durable, erosion resistant surface that would inhibit any tendency for joint faulting caused by erosion of the subbase surface for high volume traffic pavements. An econocrete subbase will also permit the paving contractor to utilize his own mixing and placing equipment for the subbase, thus spreading his move-in costs over a greater volume of work. The scheduling of the paving contractor's work would be more definite than when a subbase of a different type is sublet to another contractor. In many cases the work schedule maintained by the base subcontractor does not mesh with the paving contractor's schedule, and delays can result. The finish tolerances being obtained on some stabilized subbases have resulted in excessive concrete overruns in order to meet minimum pavement core depth requirements. These problems can be minimized with an econocrete subbase; in addition, an econocrete subbase will add considerably to the load carrying capacity of a concrete pavement structure.

In the last few years, the econocrete subbases have been used on many highways and airfield pavements (Yrijanson, 1975).

3.3 POROUS LAYERS

3.3.1 General

The concepts of pavement drainage have been recently understood by the pioneering work of Cedergren (1967) and Cedergren and Lovering (1962, 1968), though earlier Yoder (1959) had recognized that surface infiltration was a major source of water in concrete pavements. Following the concepts, the development of porous layers in highway design has been recommended. The Federal Highway Administration's "Implementation Package for a Drainage Blanket" (1972) recommends two-layered drainage blankets for removal of ground water seepage from under highway pavements. The two layer system is designed to serve as a subbased and can be described as:

1. Asphalt-treated (1.6 to 2 percent of 85-10 penetration grade asphalt cement) drainage layer (open-graded aggregate, limited to the intermediate gradation range of about 4 to 3/4 in. sieves), 4 to 6 in. thick (102 to 152 mm);

2. Filter layer of permeable material that satisfies the filter requirements, 4 to 6 in. thick (102 to 152 mm);

3. Perforated pipe under drain system.

Ring (1977) notes that even though 32 states advocate porous subbases, there is serious doubt as to whether their specified gradings permit a sufficient drainage capacity to abrogate the effect of water on performance. Subbases with up to 20 percent passing No. 200 sieve and a plasticity index of 6 are still permitted. Subbase materials with much fewer fines than this are practically impervious. Despite recognized facts, the highway engineers consider the requirements still to be in the development stage (Cashell, 1963). Very few measurements of subbase material permeability exist in literature. Barenberg et al. (1974) measured the permeability of open-graded bituminous aggregate mixture (OGBAM) at low heads in the laboratory. He also compared the behavior in prototype with an investigation of the tendency of plugging and ways to prevent it. Limited results indicate that properly designed layers of OGBAM can be used to advantage in a pavement structural system.

Currently some researchers have toyed with the idea of using porous layer concepts even for surface layers. Though this chapter is primary intended for bases and subbases, we shall discuss the top porous layers as well in this section.

3.3.2 Porous Pavements for Urban Runoff Control

Thelen et al. (1972) reports an investigation of porous pavements wherein water is intended to be passed through the pavement for the following reasons:

1. To absorb rain where it falls, preventing runoff and overflows from combined sewers and treatment plants required to handle stormwater loads;

2. To hold back stormwater surges to increase the capabilities of existing facilities to handle them.

3. To allow the filtered water to percolate at its own rate into the ground, and/or to store it for use;

4. To mitigate vehicular accidents and to minimize personal injuries on playgrounds;

5. To avoid puddles on flat surfaces (such as parking lots) and provide adequate drainage without crowns and vertical contouring;

6. To conserve water and land;

7. To present a satisfying appearance;

8. To minimize damage to the ecology; and

9. To provide a possible means of disposing of certain solid wastes.

Among the design features for the porous pavements were the following:

1. It shall carry the loads without damage;

2. It shall imbibe all or most of the rainfall on it, and water from melted snow and ice;

3. It shall survive freeze-thaw and weathering; long life is required;

4. It shall be hard to damage but easy to repair; maintenance costs must be moderate;

5. It shall not plug up; removal of trash by occasional sweeping or vacuuming may be permissible.

Among the various types of porous surfaces considered in the investigation were:

1. Various types of open-graded asphaltic concretes;

2. Porous portland cement concrete pavements (web and cover structure); and

3. Prefabricated brick assemblies.

The porous concrete pavement was not deemed feasible under the conditions tested. Shifting of the base material (soil) allowed for localized zones of high stress in the web and cover resulting in severe crack formation under cyclic loading for wet and dry conditions. A possible solution to this problem might be a stabilized base. Figure 27 shows the structure. The web openings measured 5 by 5 in (127 by 127 mm) in plan, and 4 1/2 in (114 mm) in depth. Web thickness was 2 in (51 mm); cover thickness was 2 1/2 in (63.5 mm) with 3/4-in. (17 mm) diameter holes centered over the web openings. Various types of prefabricated brick assemblies are suggested here for use but further investigations to determine relative merit are recommended.

Parking lots constructed of porous asphalt pavements have been studied in a limited way - e.g., University of Delaware, Newark (140 by 240 ft (42.7 by 73.2 m)) and Albion College, Albion, Michigan (60 by 100 ft (18.3 by 30.5 m)). Some of the disadvantages of such pavements are:

1. In regions where subgrades exhibit low permeabilities (such as the Midwest), porous pavements are not feasible since water infiltration is severely restricted. Once saturated, the soils shed instead of accept water;

2. Weakened subgrade (wet or saturated) requires increased pavement thickness to carry design loads and pavement cost increases;

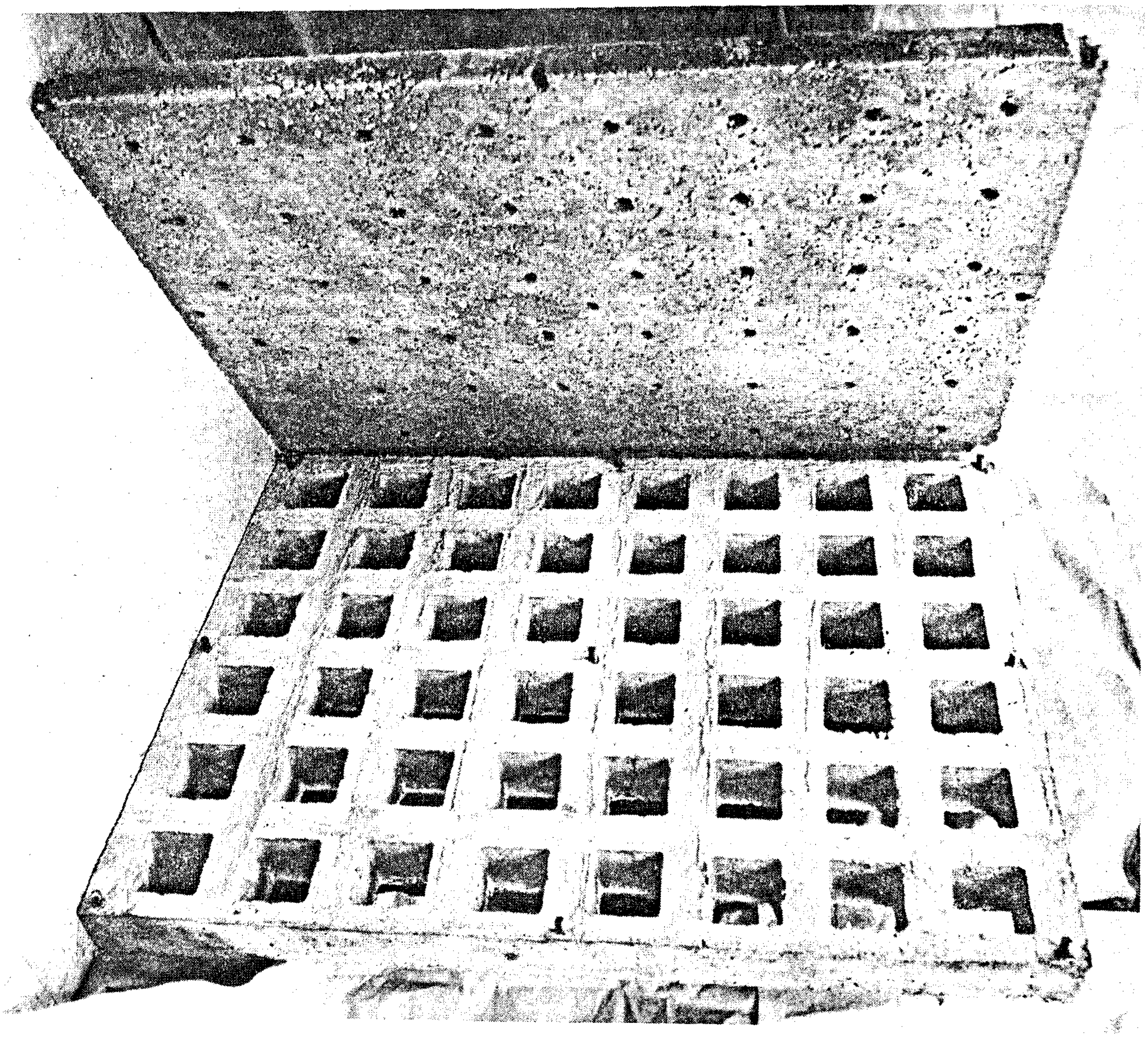


FIGURE 27 CONCRETE WEB PLUS COVER PAVEMENT

3. Clogging of pavement voids with oil and debris (maintenance problem);

4. Open-graded asphalts are weaker than dense-graded asphalts, thus requiring an increase in the open-graded asphaltic concrete thickness for a given volume of traffic;

5. For porous pavements on grade, water entering the porous layers would flow toward the low point of a vertical curve and thus emerge from the porous pavement; and

6. Incompatibility of water and asphalt resulting in a more severe stripping problem.

The potential use of porous pavements as a practical load-carrying pavement structure appears to be quite limited. At this time, the disadvantages of these pavements, including initial cost and maintenance problems, seem to prohibit their use except in certain instances.

3.4 FABRIC AND OTHER INTERLAYERS

During the past few years, a number of state highway departments and industrial site developers have looked at plastic filter fabrics to provide economy in construction costs. The growth of the filter fabrics started through their use in subsurface highway drains. At places, the underdrain pipe is even eliminated and drainage is achieved through highly permeable aggregate encapsulated in a fabric. The fabrics have also proven to be economical in also serving as a filter to prevent clogging and eliminating the expensive and time-consuming traditional sand-gravel filter. This concept opened the door for the use of filter fabric in embankments and highways. By introducing a layer of fabric between the granular subbase and the subgrade fine-grained soil, the two media are separated from each other, yet hydraulically interconnected. The filter cloth has a high permeability, and its tensile strength allows it to mold into any contour of the subgrade. Most manufacturers claim that the contribution of the fabric to a layered pavement system goes beyond just acting as a separator and a filter. They emphasize that the fabric reinforces the system (like the reinforced earth concept) (Mirafi 140, 1975). Although intuitively acceptable, the claim has neither been experimentally nor analytically confirmed to date.

Barenberg et al. (1975) conducted a qualitative evaluation of soil aggregate systems with Mirafi fabric. Though a lot of extrapolation was applied to the results of the experimental program to produce the design curves (one of them reproduced on Figure 28), qualitatively it demonstrated "that inserting a layer of fabric between the subgrade soil and the granular layer results in improved stability of the system and permits a decrease

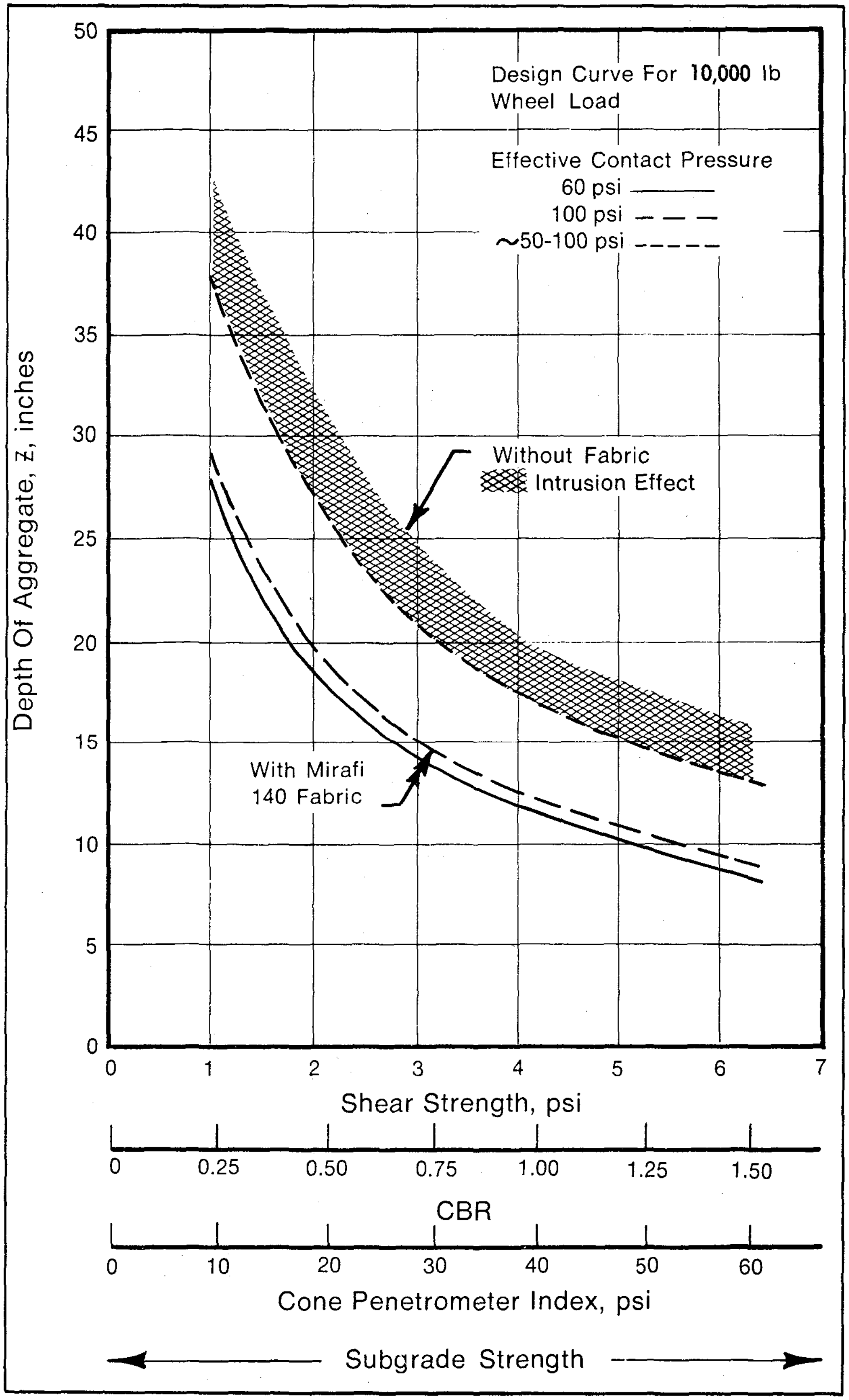


FIGURE 28 TYPICAL DESIGN CURVE FOR SOIL-AGGREGATE SYSTEMS WITH AND WITHOUT MIRAFI 140 FABRIC

in the required thickness for the granular layer. The basic function of the fabric in these systems is to keep the subgrade soil separated from aggregate material, and aid in distributing the stress transmitted to the subgrade, thus effectively improving the support conditions for the aggregate material." This later concept of "load-spreading" and/or "increasing allowable limiting stress" by use of fabrics has been stressed by Fukusumi et al. (1967) in Japan and others in France. Based on his earlier work in 1965, Barenberg feels that "a fabric, installed at soil-granular layer interface should tend to stabilize the system by increasing the interface shear strength", which is responsible for the stability of the system. Experimental work also suggests that the fabric may be effective in preventing the localized plastic flow in the soil but does not significantly alter the ultimate failure stress. The fabric layers do have a potential, but more research is required to:

1. Evaluate the strength properties of the fabric;
2. Determine the effects of the number of layers of fabric and fabric weights on the behavior of a total system;
3. Determine a better mathematical model for design; and
4. Compare the results of the analytical model with controlled laboratory tests in the first stage and with field tests in the second stage.

3.4.1 Fabrics to Control Reflective Cracking

When cracks appear on a pavement surface and become so intense that they inhibit the riding quality, the highway department, in an attempt to improve the surface, resorts to overlaying the surface with an asphaltic concrete layer. Shortly afterwards they find identical cracks appearing in the overlay, directly over cracks in the underlying pavement -- so-called reflective cracking. North Carolina is currently (Civil Engineering, May 1976) using a fabric to prevent reflective cracking. Professor William Head of North Carolina (who is working on the project) feels that the filter fabric may sometimes be an effective way to reduce reflective cracking where the underlying concrete slab is in intimate contact with the subgrade. However, in those cases where there is relative vertical movement of the concrete on opposite sides of a crack, filter fabrics may be a possible answer. Highway and Heavy Construction magazine (April 1976) reports that a polypropalene fabric has demonstrated that it can minimize or prevent reflective cracking when used between an old portland cement concrete slab and a 1.5-in. (38 mm) -thick asphalt overlay. Intuition and limited experimental work indicate that plastic fabric may be a probable method to control reflective cracking. More experiments to set limits and to examine the environmental effects on the life of plastic sheeting are required.

3.5 SYNTHETIC AGGREGATES

3.5.1 Introduction

This section presents the findings of the investigation conducted to determine the feasibility of using synthetic aggregates in untreated base coarses for zero-maintenance rigid pavement systems. This investigation excluded synthetic aggregates conventionally used, such as open hearth slag, as well as aggregates derived from crushed rock or naturally occurring gravels and sands. Chemically treated or cement and bitumen-stabilized base coarse utilizing synthetic aggregates are also excluded from this section because, except for the aggregates, the techniques are already well developed.

3.5.2 Objective and Scope

The purpose of this investigation was to search and inquire, locate, obtain, review and evaluate existing information on a limited number of synthetic and/or unconventional aggregates in the hope of identifying suitable or potential replacements for base coarse aggregates presently used. Some attention was also given to the beneficiation of low-grade aggregates.

The types of existing information incorporated in the evaluation of synthetic and/or unconventional aggregates included experimental laboratory and field tests, and performance records of prototype installations. Foreign information on the utilization of substitute aggregates that are not characteristic or representative of aggregates found in the United States, such as British reports on unburnt and/or burnt colliery shale, was not incorporated in this investigation for obvious reasons.

The evaluation itself did not incorporate the initial cost associated with the utilization of any of the candidate aggregates. The capability of providing satisfactory and maintenance-free service for a period of 20 years following installation, and requiring only minor repair in order to function successfully for an additional 10 years, was considered essential for suitable replacement aggregates. Personal communication with authorities from universities, private and state research institutes, as well as published reports and articles provided the source of information for this investigation. The following authorities are acknowledged for providing direction and/or assistance to this investigation: Dr. L.K. Moulton, West Virginia University; Dr. D. Pindzola, Franklin Institute Research Laboratories; and Mr. Robert Collins, Valley Forge Laboratories. Reports and articles on synthetic aggregates were also located using the Engineering Index Manual, The John Crerar Library and the U.S. Federal Highway Administration - Department of Transportation Library card catalogs and also the National Technical Information Service's computerized data base.

3.5.3 Summary of Findings

Seven types of synthetic aggregates, which can be categorized under two basic sources, are reviewed and evaluated. The aggregate types and sources are illustrated on Figure 29. A third source of aggregates derived from the beneficiation of aggregates obtained from the processed waste materials and/or the heat-treated materials is also illustrated on Figure 29. Based on available information, it appears that suitable replacements do not presently exist for major pavement systems subjected to intense and extreme loads and/or severe climatic conditions. Several potential replacements, however, are identified and may prove to be suitable subsequent to additional research and testing. The present unavailability of acceptable synthetic aggregates for untreated base coarse applications does not appear to be a serious problem if the initial cost of the zero-maintenance pavement system is considered to be irrelevant. Alternatives such as the importation of high quality conventional aggregates, sophisticated quarrying or mining processes, and treatment methods such as chemical stabilization may then be proposed for obtaining the desired base coarse performance.

3.5.4 Review of Literature

Due to the rather recent concern for supplementing conventional aggregates in all types of highway and pavement systems applications, and also industrial concerns regarding the economic justification of synthetic aggregate development and production, a limited amount of engineering data useful in analyzing and evaluating synthetic aggregates is available. Data relating to the utilization of synthetic aggregates in base coarses, as opposed to its utilization in concretes and surface coarses, are even more limited due to the economic implications. In the past and also at present, the top dollar in pavement systems is associated with the wearing coarse.

Marek et al. (1972) compiled a report that in part presents the proceedings of a specialty conference held in 1970 for reviewing and predicting the needs of highway engineers concerning aggregate utilization in highway construction. The report is the second of such (NCHRP Report No. 8, 1974) sponsored by the Highway Research Board to provide an update on the needs, developments and required research regarding synthetic aggregates. In a review of existing available conventional high quality construction aggregates, the southeastern United States, especially along the Gulf Coast, had the most limited supply. Engineering properties and/or characteristics of previously investigated synthetic aggregate for use in base coarse analysis and design are not given. Better utilization of existing conventional aggregates is believed necessary for meeting anticipated demands in the near future. Possible

1) Processed Waste Materials
   a. Bottom Ash
      i) dry bottom ash
      ii) wet bottom ash (boiler slag)
   b. Zinc Smelter Waste
   c. Spent Oil Shale

2) Heat Treated Materials
   a. Clays (nonbloated)
      i) naturally occurring
      ii) mixtures
   b. Marine Muds
      i) silty dredge spoil
   c. Fused Garbage Residue
      i) "Eco-Rock"

3) Beneficiated Materials
   a. Mechanically Processed
      i) washed
      ii) separated (heavy media)
      iii) crushed
      iv) sieved
   b. Blended
   c. Coated
   d. Impregnated

FIGURE 29

SUPPLEMENTAL AGGREGATE TYPES REVIEWED

sources for supplemental aggregates intended to help meet the increasing demands, which may also simultaneously offer some ecological benefits, are presented. These sources are illustrated on Figure 30, and lend themselves to being reduced to the following six broad categories (NCHRP Report No. 135, 1972):

1. Heat treatment of raw minerals;

2. Chemical and thermochemical processing of raw minerals or solid wastes from conventional aggregate production;

3. Pelletizing of raw minerals and beneficiation to suitable quality;

4. Utilization of solid wastes;

5. Beneficiation of low quality aggregates; and

6. Utilization of marine deposits.

Aggregates produced utilizing four of these six methods are presented, discussed and evaluated in subsequent sections of this chapter.

General research was proposed for some 14 topics, each topic requiring a period of investigation of approximately 4 to 5 years after its initiation. With delays in awarding contracts, lack of funds, redefinition of scope, etc., it is expected that most of the information resulting from these studies on synthetic aggregates is not yet completely available. Some preliminary results seem to indicate that a revision of the existing material and construction specifications as well as their associated test procedures is in order for adaptability to the sometimes peculiar but not necessarily detrimental characteristics of synthetic aggregates.

Only the most relevant findings of this investigation are presented in the following section due to the inherent nature of this study. Summaries with appropriate references to the original works are presented since the information upon which the evaluation is based is well documented and readily available. Also included in the references are some reports concerned with testing of conventional aggregate materials. These have been only briefly reviewed in the hopes of gaining some additional insight into desirable characteristics of base coarse aggregates as well as innovative test procedures. Information obtained from these reports is not explicitly stated in this chapter and has only been incorporated in a very general sense in the evaluation of supplemental aggregates. These data will be used in the analysis of pavement systems under Task B of this contract, since these pavement systems are expected to contain an integration of both conventional and innovative components.

1. Heat treatment of suitable clays and shales

2. Pelletizing and sintering of raw materials

3. Pelletizing of raw materials and coating

4. Mechanical processing of certain wastes

   a. Building and highway rubble

   b. Ceramic waste

      1) Devitrified waste
      2) Glass cullet
      3) Waste ceramic ware

   c. Waste from cast-iron enameling

   d. Wastes from structural clay products

      1) Broken bricks
      2) Broken pipe
      3) Broken tile

   e. Metallurgical slags

      1) Incineration of sewage sludge
      2) Incineration of garbage

   f. Scrap iron and steel

5. Heat treatment of nonbloating materials including sand, soil and loess

6. Manufacture by nuclear methods

7. Heat treatment of waste materials

   a. Coal mine tailings

   b. Fly ash

   c. Collected mineral dusts

8. Chemical and thermochemical processing of raw materials or solid wastes from conventional aggregate production

   a. Mixing with agent, pelletizing and curing

      1) Lime
      2) Lime-fly-ash
      3) Cements (phosphate, silicate, portland, etc.)
      4) Plastics

   b. Mixing with agent, casting brick, curing and crushing to size

FIGURE 30

EXISTING POTENTIAL SOURCES FOR THE PRODUCTION OF SUPPLEMENTAL AGGREGATES
(From NCHRP Report No. 136, 1972)

9. Mechanical processing of marine deposits
    a. Reef shell
    b. Beach sand
10. Beneficiation of a low-quality aggregate
    a. Coating
    b. Impregnation
    c. Blending
    d. Mechanical processes
        1) Washing
        2) Separation (heavy media)
        3) Crushing
        4) Sieving
11. Combination of two or more of the previously listed sources

FIGURE 30 (continued)

3.5.4.1 Criteria

Factors considered essential for base coarses of rigid pavement systems intended to provide adequate stability and drainage (Yoder, 1975) are outlined as follows:

1. Control of Pumping

    a. High rainfall and/or sufficient ground water: open-graded aggregate with filter;

    b. Low rainfall and low ground water levels: relatively dense, well-graded aggregate;

2. Control of Frost Action

    a. Non frost-susceptible aggregate

        i) low absorption;
        ii) very little or not fines;

    b. Free draining;

3. Drainage

    a. High permeability:

    very little or no fines;

4. Strength

    a. Hard (nonfriable), durable and angular aggregate

        i) short-term for support of construction equipment;
        ii) long-term for load distribution;

5. Control of Shrink-Swell of Subgrade.

It is also considered desirable but not essential for the base coarse to provide, without additional treatment, a level surface upon which the wearing coarse can be constructed.

3.5.5 Processed Waste Materials

The utilization of waste materials as road construction aggregates has received considerable attention in recent years, partly due to the popularization of recycling. The usefulness of recycled wastes in pavement base coarse applications is dependent not only on the engineering properties of these materials, such as strength, durability and permeability, but also on the compatibility of the materials with the environment in which they are used. It is the intent of this investigation, however, to concentrate on the acceptability of all candidate materials as possible base coarse aggregates with respect to their engineering characteristics.

3.5.5.1 Bottom Ash

The bottom ash referred to herein is a by-product from the burning of pulverized bituminous coal in steam generating power plants. Two basic types of bottom ash are produced in the United States and are commonly designated as boiler slag (also referred to as wet bottom ash) and dry bottom ash or cinders. The type of bottom ash produced depends on the boiler type utilized by the power plants. The quality and uniformity of the bottom ash are dependent on the coal source, degree of coal pulverization, burning temperatures and also the load on the plant (Anderson et al., 1976).

Over 90 percent of the coal used annually in the United States is in the area west of the Mississippi River (Seals et al., 1972). It has also been estimated that 25 percent of all power plant ash produced in the United States is boiler slag, 10 percent is dry bottom ash and 65 percent is fly ash (Seals et al, 1972). It is anticipated, however, that dry bottom ash production will increase in the future (Anderson et al., 1976).

Characteristics - A preliminary study was conducted by Seals et al. (1972) to determine some of the physical, chemical and engineering properties of bottom ash. Results from tests on both types of bottom ash produced at several different locations in West Virginia are presented in the report for this study. Although properties tend to vary considerably depending on the source of the bottom ash, some insight can be obtained into the general characteristics of both types of bottom ash. Anderson et al. (1976) also presents typical properties of two types of bottom ash as shown in Table 10. Additional observations are presented separately for each type of bottom ash and are as follows:

A. Boiler Slag

1. Hard, angular to subangular with some rod-shaped particles having a glassy texture;

2. Poorly graded ranging in size primarily from fine gravel with some oversized material and also non-plastic fines;

3. Dry density of 90 pcf (1442 kg/m^3) (based on one rodded air sample from Seals et al., 1972);

4. Relatively little degradation during Standard Proctor Compaction (tests results are similar to those obtained for many sands);

TABLE 10

ENGINEERING PROPERTIES OF POWER PLANT AGGREGATES

SOURCE OF ASH (W. Va.)	TYPE OF ASH	BULK SPECIFIC GRAVITY[a]	PERCENTAGE OF WATER ADSORPTION[a]	LA ABRASION[b]	$MgSO_4$ SOUNDNESS[c]	FRIABLE PARTICLES[d]	FLORIDA BEARING VALUE	ANGLE OF INTERNAL FRICTION (DEG)
Fort Martin	Dry	2.31	2.0	30 to 45	15	yes	198	40
Mitchell	Dry	2.68	0.3	30 to 40	10	no	--	43
Kammer	Wet	2.76	0.3	37	10	no	72	41
Willow Island	Wet	2.72	0.3	33	15	no	46	42

[a] ASTM C 127

[b] ASTM C 131

[c] ASTM C 88

[d] ASTM C 142

5. Coefficient of permeability of approximately 100 ft/day (30.5 m/day) at a void ratio of 0.88; and

6. At low stress levels, the compressibility is similar to that of sand placed at the same relative density (D_r) (i.e., no noticeable effect upon saturation;

B. Dry Bottom Ash

1. A mixture of hard and soft angular particles with a very porous surface texture;

2. Relatively similar in size to boiler slag but well graded (nonplastic fines and also no oversized materials);

3. Dry density from approximately 50 pcf to 100 pcf; (801 to 1602 kg/m^3);

4. Susceptible to degradation during Standard Proctor Compaction, otherwise similar to boiler slag;

5. Slightly lower permeabilities than boiler slag; and

6. Similar compressibility to boiler slag.

The primary differences between the bottom ashes are the gradation, surface texture and particle hardness. These differences are worth noting since they directly relate to the stability and permeability, durability and strength of base coarses utilizing bottom ash.

Performance - Bottom ash has been utilized in untreated bases for several different pavement systems in West Virginia and several surrounding states (Moulton et al., 1973), although national utilization of bottom ash still lags production substantially. This limited experience indicates that bottom ash utilized in bases without screening or additional treatment loses stability during construction operations upon drying out. This loss in stability in an unconfined state can be remedied by mixing the bottom ash with approximately 40 percent (by weight) of fly ash. More than adequate required in-place densities are obtained through vibratory compaction at "wet-of-optimum." In increasing the stability according to this procedure, one in turn then reduces the drainage capabilities of the base and increases the susceptibility to pumping at pavement joint locations.

Beneficiation - Screening of the bottom ash and mixing with other aggregates can result in acceptable base coarses regarding stability. Instances in which the bottom ash was mixed with coarse blast furnace slag aggregate have been known to produce

Class 1 type base coarses (Moulton et al., 1973) for West Virginia pavement systems. Additional treatment of bottom ashes may also be necessary on occasion, prior to utilization in base coarses, to purge the aggregates of contaminants such as pyrite and sulfate introduced into the coal source, thus preventing aggregate degradation upon exposure to water for extended periods of time and also protecting adjacent concrete structural systems (Anderson et al., 1976).

Evaluation - It is concluded that bottom ash of both the dry and wet variety shows some promise for use in zero-maintenance pavement systems. A more complete understanding of the behavior of this material, obtainable through continued laboratory and field investigations, is essential.

An effort should be made to obtain some conclusive data on the freeze-thaw characteristics of both bottom ash types, to assess the performance under harsh climatic conditions. Severe climatic conditions prevail in more than half of the United States wherein the bulk of this material is produced and would therefore be used. Large-scale tests utilizing representative samples of stable base coarses should be conducted to determine their drainage characteristics.

Due consideration must be given to the variability in the properties of untreated bottom ash primarily resulting from the utilization of numerous coal sources, coal additives for meeting efficiency and/or environmental demands and also stockpiling practices. Excessive testing to insure conformance with standard specifications might therefore prove necessary.

Based on the information currently available, bottom ash seems to be best suited for use in subbases of zero-maintenance pavement systems where gradation and strength requirement are less critical. In this manner, better utilization of conventional high-grade aggregates in more demanding applications is permitted.

3.5.5.2 Spent Oil Shale

Spent oil shale is the by-product of retorting processes on raw oil shale for the production of crude shale oil. The retorting process may consist of indirectly heating raw oil shale, a fine-grained and compact sedimentary marlstone, between temperatures of approximately 700° and 950°F (371° and 510°C) in order to vaporize the entrapped organic substance known as kerogen (Gromko, 1975). Subsequent condensation of the vapors results in crude shale oil; Colorado, Wyoming and several other surrounding states are rapidly accumulating enormous waste piles of spent oil shale from the oil extraction processes. Methods utilizing spent oil shale, ranging in size from chunks as large as 20 in. (508 mm) to particles smaller than 0.0029 in. (0.07 mm, minus No. 200 sieve) are being sought. Past research efforts have concentrated primarily on the utilization of spent oil shale passing the No. 200 sieve, commonly referred to as spent oil shale ash. A recent preliminary investigation has focused on the utilization of spent oil shale aggregates (maximum size of 3/4 in. (19 mm)) obtained from the U.S. Bureau of Mines, Laramie Energy Research Center (LERC), in Wyoming and originating from the Green River Formation (Gromko, 1975).

Characteristics - Results of tests obtained from previous investigations on spent oil shale ash, as well as results obtained by Gromko (1975), are presented by him; aggregate characteristics such as size, grading, toughness, particle shape, surface texture, absorption and strength are included. General observations of the aggregate properties are presented below (observations are based on samples prepared by sieving and grading as shown on Figure 31.

A. Spent Oil Shale Aggregates

1. Flat, friable and angular, having a very porous and rough surface texture;

2. Well-graded material possible upon crushing and sieving (nonplastic fines);

3. Dry density of 85 pcf to 95 pcf (1362 to 1522 kg/m^3;

4. Low abrasive resistance;

5. Very high percentages of water absorption; and

6. Angle of internal friction

   a) moist, $\phi = 35^{\circ}$

   b) saturated, $\phi = 20^{\circ}$.

Performance - To our knowledge, spent oil shale aggregates have not been incorporated in field installations of pavement systems.

Beneficiation - Crushing and sieving of the spent oil shale is necessary prior to its use in base coarses. Since the properties of spent oil shale vary considerably with temperature utilized in the retorting process, it seems feasible to alter and possibly improve the aggregate characteristics through heat treatment (burning) at high temperatues.

Evaluation - Based on the limited scope of work in his investigation, intended to assess the feasibility of spent oil shale aggregate utilization in flexible pavement systems, Gromko (1975) highly recommends the use of these aggregates as conventional base coarse aggregates. This recommendation is based on the high strength achieved by the dense well-graded samples of moist spent shale aggregates.

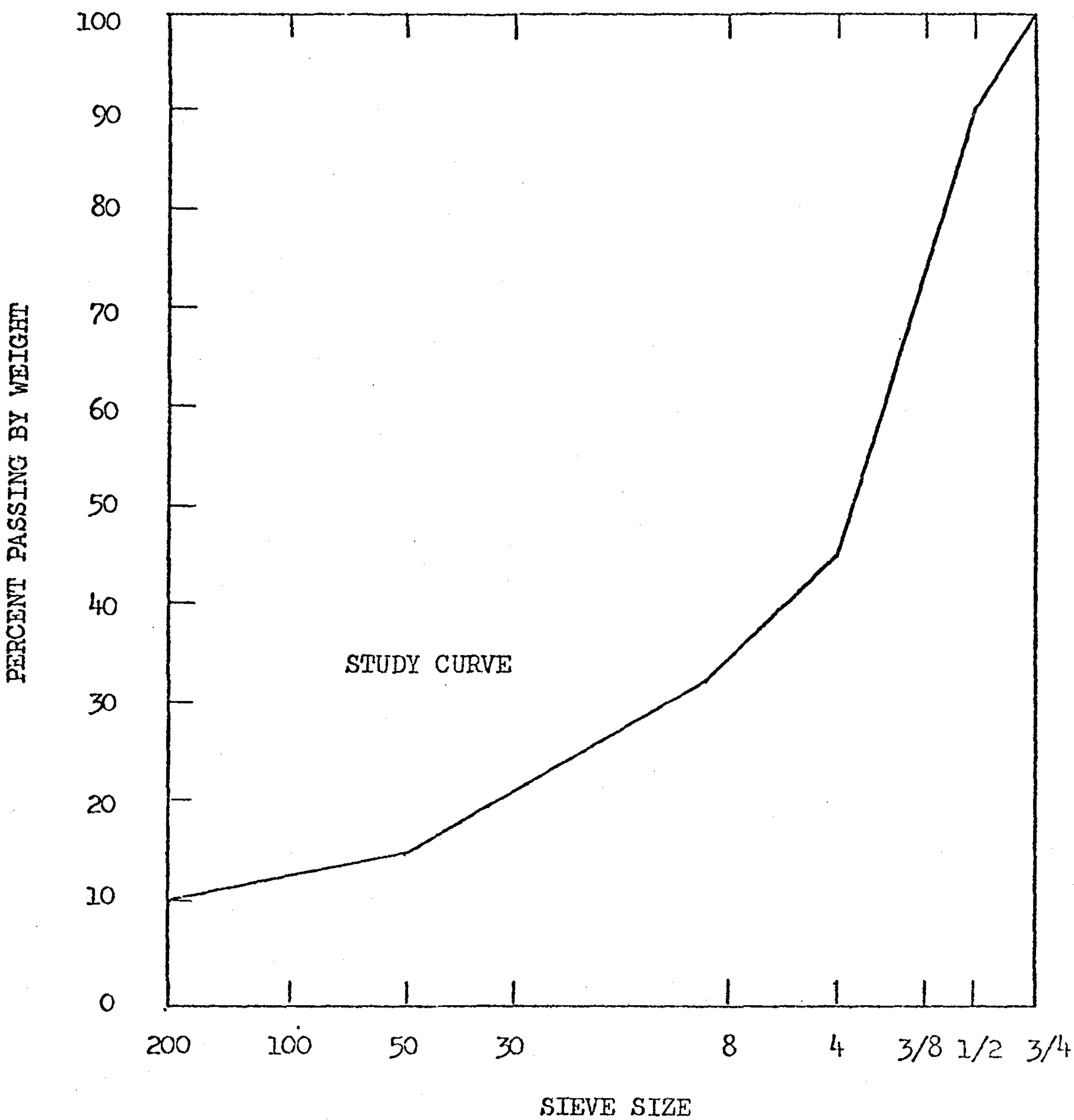


FIGURE 31 GRADATION CURVE FOR SPENT OIL SHALE AGGREGATE (FROM GROMKO, 1975)

We are not in agreement with this blanket recommendation, and more specifically do not feel the particular aggregate investigated to be suitable for general use in zero-maintenance pavement systems based on the information presented. The following reasons are cited:

1. Friable particles subject to degradation during handling and compaction, therefore altering the gradation;

2. High absorption capacity indicative of limited durability when subjected to alternate cycles of freeze-thaw resulting in aggregate break-up; and

3. Strength loss upon saturation (possible from infiltration of water through the wearing coarse and/or joint openings and high ground water tables) resulting in the reduction of load carrying capacity (during or after construction).

Possibilities might exist, however, for the utilization of spent oil shale aggregates in subbases of pavement or in base coarses upon some form of beneficiation as mentioned previously. Additional investigations, both in the field and laboratory, should be conducted to evaluate the performance of this material more completely.

3.5.5.3 Zinc Smelter Waste

Oklahoma has available large quantities of smelter waste resulting from mining operations. Hughes and Halliburton (1973) conducted a study to determine the physical and engineering properties of zinc smelter waste necessary for evaluating its feasibility for use as highway construction material. Of particular interest to this investigation are their findings regarding mechanically stabilized base coarses.

Characteristics - Samples for testing purposes were obtained from four types of Oklahoma zinc smelter waste. General properties of these materials are the following:

1. Brittle as well as durable cubical particles with a sharp angular and very porous surface texture;

2. Relatively uniformly graded and void of coarse particles (majority of particles smaller than No. 10 sieve);

3. Specific gravity of 2.18 to 3.14;

4. Water absorption (%) of 3.4 to 5.0;

5. High alkali reactivity;

6. Subject to leaching of soluble zinc; and

7. Angle of internal friction of 47° to 54°.

Performance - To our knowledge, zinc smelter wastes have also not been incorporated in field installations of pavement systems.

Beneficiation - The sieving of zinc smelter waste and recombination with other coarse aggregates is necessary to meet particular unwanted zinc leachates (damages or kills vegetation) and adverse reactivity between zinc smelter waste and adjacent unprotected portland cement concrete (PCC).

Evaluation - The types of zinc smelter wastes discussed above are not applicable for use in their natural state in stable free draining base coarses. However, based on the preliminary information available, zinc smelter wastes may be considered to be an excellent potential source of fine aggregates for use in mechanically stabilized base coarses.

3.5.6 Heat Treated Materials

Three types of synthetic aggregates produced through the thermal transformation of highly plastic clays, incinerator residue and marine silt are evaluated. The thermal transformation of materials in the United States is accomplished utilizing two basic methods; namely, the rotary kiln method and the sintering method (Ledbetter, 1973).

3.5.6.1 Fire Clay Aggregates

Moore (1970) presents the results of several tests on non-bloated synthetic aggregates produced from naturally occurring highly plastic clays. The clays were transformed into inert, stable, amorphous silicates under the combined action of heat and time in a commercial rotary kiln and also in the Texas Transportation Institute's (TTI) research rotary kiln. Criteria for establishing the acceptability of these synthetic aggregates for use in flexible bases and an evaluation of the performance of several roads utilizing flexible bases are also presented.

Although the investigation by Moore (1970) deals primarily with the use of fired clay in flexible bases, several observations can be made regarding the use of these aggregates in rigid pavement base coarses.

Characteristics - Laboratory test samples produced in the TTI kiln (Moore, 1970) were prepared from three types of naturally occurring plastic clays found in Brazos County, Texas (Table 11). Four additional aggregate samples, produced commercially and previously used in pavement base construction, were also tested. Firing temperatures for TTI aggregates were deliberately varied from as low as 1000°F (543°C) up to 2000°F (1094°C) to determine their influence on aggregate properties. The following represent the most pertinent observations:

TABLE 11

MATERIALS USED BY MOORE (1970) FOR
RESEARCH AGGREGATE PRODUCTION

RAW MATERIAL	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	GRADATION*		
				PERCENT SAND	PERCENT SILT	PERCENT CLAY
Red Clay	74	28	46	4	24	72
Gray Clay	67	23	44	5	41	54
Black Clay	53	19	34	13	34	53

*Determined by hydrometer analysis MIT Classification sand, 2 to 0.06 mm; silt, 60 to 2 microns; and clay, smaller than 2 microns.

1. The clay minerals montmorillonite, illite and kaolinite can be used in most cases to produce high quality synthetic aggregates by completely dehydrating them; once completely dehydrated they will not rehydrate;

2. The behavioral characteristics of fired aggregates vary considerably when firing temperatures are raised from 1000° (534°C) to 1500°F (816°C). Firing temperatures in excess of 1500°F (816°C) up to 2000°F (1094°C) do not appreciably alter aggregate characteristics;

3. Sound, hard aggregates may be obtained at firing temperatures of approximately 1500°F (816°C) for the clays tested;

4. Gradation seems to be lacking (95 percent of the resulting aggregates were between 1 in. (2.54 cm) and the No. 10 sieve in size;

5. None of the aggregates classified as lightweight aggregates (the loose unit weight exceeded 55 pcf (881 kg/m^3));

6. Fired clay aggregates exhibit relatively high water absorption capacities (Ledbetter, 1973);

7. Good chemical stability (low reactivity with PCC) is achieved; and

8. The aggregates are susceptible to degradation as a result of conventional compaction procedures and severe climatic conditions. Experience has indicated that the use of pneumatic rollers reduces the degradation during field compaction (Horn, 1967).

More detailed information of fired clay aggregate properties is presented by Moore (1970), Moore et al. (1966) and Houston (1969).

Performance - Fired clay aggregates produced in commercial kilns have been used in combination with approximately 30 percent (by weight) of field sand (water content <35 percent, plasticity index <10) in several Texas roads (Moore, 1970). The field sand is added to meet the necessary gradation requirements. The commercially produced aggregates were fired at approximately 1400°F (760°C). Good performance was experienced over a service period of approximately 6 years at the time of the investigation. No appreciable disintegration had resulted, as determined from laboratory tests on samples obtained from these roads. Load and traffic frequency data imposed on these roads were not presented.

Beneficiation - Although not presented in the literature reviewed for this investigation, several beneficiation techniques other than mechanical processing might prove feasible in enhancing the quality of fired clay aggregates. Coating or impregnation of fired clay aggregates, possibly with polymer-type materials, would essentially eliminate the high water absorption capacities, therefore rendering the aggregate more durable (less frost susceptible). Blending the clay minerals with certain materials prior to the heat treatment process might also produce a more durable product.

Evaluation - Fired clay aggregates show some promise for use in base coarse applications for zero-maintenance pavement systems based on the preliminary information reviewed, especially in moderate climates. More information on the properties and performance of these aggregates needs to be developed and/or accumulated, however, in order to present a more convincing argument. Lytton (1977) is of the opinion that when mixed according to proper proportions, montmorillonite, illite and kaolinite can form the basis for a hard and durable fired aggregate suitable for use in untreated coarses.

3.5.6.2 Fused Incinerator Residue

The aggregate that is referred to herein is produced utilizing a fusion process designed to transform incinerated municipal refuse into paving materials. The resultant fused incinerator residue is trade named Eco-rock. Dr. Daniel Pindzola of the Franklin Institute Research Laboratory, initially funded by the U.S. Public Health Service and presently by the Federal Highway Administration's Department of Transportation, is the designer of the process. The aggregate production process basically consists of the following steps (Pindzola, 1972; Pit and Quarry, 1976):

1. Screening incinerator residue to <1-in. (2.54 cm) size (the residue represents approximately 16 percent of the original municipal refuse);

2. Introduction into a rotary kiln for 15 minutes @ 1600°F (871°C) to 1800°F (982°C);

3. Combustion @ 2000°F (1094°C) in a chamber to accomplish fusion;

4. Cooling to a solid state, approximately 700°F (371°C); and

5. Crushing and screening of aggregate to desired gradation.

Engineering properties useful in evaluating the aggregate for suitability in rigid pavement base coarses are not yet believed to be available. The utilization of Eco-rock as aggregate for wearing coarses has been the major objective of previous investigations (Pindzola, 1977). Excellent skid resistance is exhibited by the aggregates. Good handling characteristics are also exhibited by the aggregates.

Performance - No performance records of Eco-rock aggregate in pavement systems have been obtained.

Evaluation - Due to the lack of available technical information on aggregates produced from Eco-rock, an evaluation is at present not possible. Its utilization in base coarses does appear to have considerable potential, however, in light of its apparent acceptability as surface coarse aggregate. Eco-rock is presently being produced at a pilot plant in Pennsylvania under a U.S. Department of Transportation - Federal Highway Administration contract. Information on any testing and developments of Eco-rock should be obtained whenever made available and subsequently reviewed to assess its suitability in all types of pavement application. Aside from its potential for relieving the burden on conventional aggregates utilized in pavement construction, Eco-rock also offers an attractive solution for waste disposal.

3.5.7 Sintered Dredge Spoil

Dredge spoil consisting of fine-grained marine mud recovered from harbors and embayments along the northeast coast of the United States was sintered for approximately 4 hours at 1920^{o}F (1050^{o}C) to produce aggregates investigated for use as a construction material (Rhoads, 1975).

Characteristics - The strength-to-weight ratio of the aggregates produced from sintered mud is comparable or superior to commercially used lightweight expanded shale aggregates. Crushing strengths of sintered silt aggregates were measured in a 3.0-in. (7.6 cm) by 5.1-in. (13 cm) steel compression cell. The aggregate size tested ranged from the No. 16 sieve (1.10 mm) to the No. 4 sieve (4.76 mm). Aggregates consisted of equiaxial particles, and exhibited rather high permeabilities (magnitude not reported). Crushing strengths reported for both sintered marine muds and expanded shales by Rhoads (1975) are presented in Table 12. Rhoads (1975) determined that the sintered aggregates with the best properties are made from feed containing 50 percent illite and chlorite with lesser amounts of other clay minerals. He also established that the increase in the quality of shell carbonates and/or sands in the feed resulted in rapid aggregate strength degradation.

Performance - Applications of these types of aggregates in pavement systems are not reported by Rhoads (1975) and are presently unavailable.

Evaluation - Additional information on the physical and engineering properties of these aggregates is necessary before any valid conclusions can be made with respect to their use in rigid pavement base coarses. Some positive as well as negative observations regarding their utilization in base coarses are the following:

TABLE 12

CRUSHING FORCE REQUIRED TO COMPRESS AGGREGATE 2.5 cm (S_1) AND 5.0 cm (S_2) IN COMPRESSION CELL

(From Rhoads, et al., 1975)

MATERIAL[a]	d, g/cm^2	S_1, psi	S_2, psi	S_2/d[b]
Sintered silt A	1.70	714	11,430	6,700
Expanded clay R_1	1.80	890	4,100	2,300
Expanded clay R_2	2.28	140	610	260

[a]Silt A is a Cape Cod Bay mud; aggregates R_1 and R_2 are expanded shales.

[b]The S_2/d ratio of the Cape Cod Bay aggregate is, respectively, 3 and 26 times that of the expanded shale ratios.

(1 psi = 0.0703 kg/cm^2)

1. Relatively large crushing strengths indicating good load distribution characteristics;

2. Control of particle shape and gradation enhancing the field stability (unconfined condition);

3. Noncontinuous void structure due to sintering process resulting in reduced absorption and possibly increased durability;

4. Increased aggregate size resulting in decreased strength due to greater chance of shell carbonate and/or sand particle inclusion;

5. Possible treatment of aggregate required to insure prevention of leaching of soluble compounds introduced by contaminants in the feed materials; and

6. Excessive handling and preparation as well as transportation of feeder material and/or aggregates.

3.5.8 Conclusions and Recommendations

The evaluations of aggregates reviewed in this investigation for possible use in zero-maintenance rigid pavement base coarses have been presented separately under each aggregate type. At present, it may be concluded that based on the information available and reviewed for this investigation, none of the synthetic aggregates considered appear to be acceptable for their intended use. Several aggregate types do, however, exhibit a potential for future use. Additional information regarding synthetic aggregate properties and performance, when applicable, needs to be determined and/or accumulated. More information from on-going research projects should become available in the near future. No new methods, other than those presented above, appear to be forthcoming in the near future for the production of synthetic aggregates. Better utilization of conventional aggregates as well as the revision of existing specifications is believed necessary to meet future aggregate demands.

3.6 CHEMICAL SOIL STABILIZATION

The use of additives, such as cement, lime, lime-fly ash, bitumen, and other organic and inorganic chemicals, for the construction of pavement structures has been studied and used extensively in the past 40 years. The additives, when properly mixed in and compacted with locally available materials, may improve the strength, volume change, water proofing, and/or freeze-thaw characteristics of the materials for use as a pavement subbase or base course. Comprehensive reviews and bibliographies dealing with these stabilization techniques may be found in the Transportation Research Board publications.

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CHAPTER 4

THE SURFACE

4.1 INTRODUCTION

Scientists consider it respectable to define the rationale of material behavior on a molecular level, although the main structural aluminum alloy of the Concorde supersonic airliner was developed in the early 1930s, well ahead of the electron microscope that can reveal the alloy's structure. Despite all the contemporary aids of science, the exact molecular process in the conversion of polymeric fibers to high modulus carbon fiber is still a subject of some speculation nearly a decade after the original discovery even though the fiber is moving towards large-scale production. Nevertheless, the people concerned with these inventions know what properties to look for. This chapter, therefore, will consider the innovative materials and concepts forming the surface of the pavement from the point of view of their suitability for zero maintenance. The materials reviewed include fiber and polymer reinforced concretes. The systems studied are prefabricated, waffle, grill and corrugated. The suitability of pavement on piles is also discussed. Relevant discussions on innovative joints, composite systems and related topics are also presented.

4.2 FIBER REINFORCED CONCRETE

4.2.1 Background

When considering an innovative pavement-surface material, the energy required per unit area cracked open can be related to real engineering situations. The possible relationship between work of fracture (fracture energy), applied stress and permissible crack length must be determined. Griffith (1920) first derived the relationship for brittle materials (glass), and his work has been generalized to cover other materials. Fracture energy is measured in joules per square centimeter (J/cm^2) of crack area produced. Appendix I provides a short description of how Griffith related fracture energy and crack length to the stress needed to produce catastrophic failure. Fracture energy in itself does not reveal directly what is happening in the material. Griffith noted that maximum attainable stress varies inversely as the square root of crack length and that materials of low fracture energy could tolerate only very small cracks. That is, the crack length needs to be reduced by a factor of 10^4 to make such materials 100 times stronger. Small cracks (flaws) are present in all materials and failure occurs by propagation of a crack from largest flaw. The larger the specimen, the more likely it is to contain a large flaw and hence to have a low strength. If thin fibers (having very little flaws) are embedded in a proper matrix material, it is possible to utilize the strength of virtually all the fibers. The mixture of the fiber and matrix is effective in deflecting cracks where it separates two phases of differing modulus, and a mechanism is evolved in frustrating the crack propagation rather than increasing fracture energy per se. The above background discussion is to clarify the concept that the role of fiber reinforcement is not really to improve static strength but to control cracking. The controlled cracking increases the ductility, energy absorption, and resistance to impact and thermal loading of the composite material. Effective control of the size of cracks may make the composite less vulnerable to environmental effects. A brief discussion of the mechanism of fiber reinforcement is presented in the following section.

4.2.2 Mechanics of Reinforcement

In this section, the results of putting stress fibers into a matrix are considered. The matrix should be effective in transferring stress from fiber to fiber, so that applied loads may be evenly distributed, but without the ready propagation of cracks that would occur in a homogeneous brittle solid. Despite the fact that extensive mathematical literature on composites exists, the observed properties do not justify more than an approximate theory. This is because the basic theoretical unit is a fiber rather than an atom and therefore warrants a macroscopic rather than a microscopic treatment.

The influence of fibers on the elastic properties of the composite can be approximated by the following equation (see Figure 32):

$$E_c = \alpha E_f + (1-\alpha)\, E_m \qquad \text{(Equation 4-1)}$$

where

E_c = modulus of composite

E_f = modulus of fiber

E_m = modulus of matrix

α = volume fraction, or properties of cross section, occupied by fibers.

This simple proportion law is sometimes called the law of mixtures. If the matrix exceeds the elastic limit, its stress is still written:

$$\sigma_c = \alpha\sigma_f + (1-\alpha)\, \sigma_m \qquad \text{(Equation 4-2)}$$

where

σ is the stress and subscripts c, f and m denote composite, fiber and matrix

but

$$\sigma_m \neq E_m \varepsilon$$

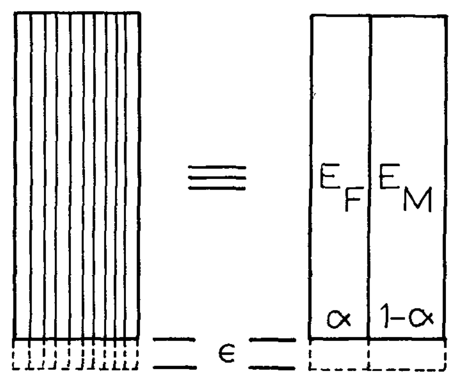


FIGURE 32 LAW OF MIXTURES FOR UNIDIRECTIONAL COMPOSITE

The secondary corrections of elasticity due to differing Poisson's ratio of fiber and matrix being small are neglected.

In general, the load carried by the fibers at fracture greatly exceeds that which can be achieved by the pure matrix, so that Eq. 4-2 applies with the matrix stress at fiber fracture strain. If the fibers do break together but the matrix does not, the weakening effect at volume fractions (of fiber), will be described by:

$$\sigma_c = (1-\alpha)\ \sigma^1_m \qquad \text{(Equation 4-3)}$$

where σ^1_m is the matrix strength.

The effect is shown graphically on Figure 33, the minimum strength occurring when Eq. 4-2 = Eq. 4-3. One can deduce the breakdown point for the volume fraction at which the strengthening commences by:

$$\alpha_{crit} = (\sigma^1_m - \sigma_m)/(\sigma_f - \sigma_m) \qquad \text{(Equation 4-4)}$$

The above equation suggests that if the fibers are short enough, they will be unable to reach their breaking stress again.

In the case of a unidirectional model, mechanical properties at angles other than the fiber axis can be relatively poor. Depending on the matrix and even on strength properties of unidirectional composition, there is considerable scatter (Dimmock et al., 1969). In actual fiber-reinforced materials or composites, the slight misalignment of fibers with each other can grossly alter the interlaminar shear strength, which may be a favorable effect. To account for arbitrary orientation, Eq. 4-1 can be modified (Krenchel, 1964) as:

$$E_c = C_r\ \alpha\ E_f + (1-\alpha)\ E_m \qquad \text{(Equation 4-5)}$$

The correction factor C_r varies from 1 to zero depending on the fibers being parallel to the force or perpendicular to the force. For fiber distributed in all directions, a value of $C_r = 0.2$ was recommended. The above equation can also be written as:

$$E_c/E_m = 1 + \alpha\ (nC_r - 1) \qquad \text{(Equation 4-6)}$$

where $n = E_f/E_m$, the modular ratio.

Eq. 4-6 shows the contribution of both the volume fraction and the modular ratio on the composite and matrix behavior.

It is now almost accepted (ACI, 1973) that the composite material approach describes the entire behavior of fibrous concrete until failure. To understand the transverse strength of a composite, consider a unidirectional composite and a very simple

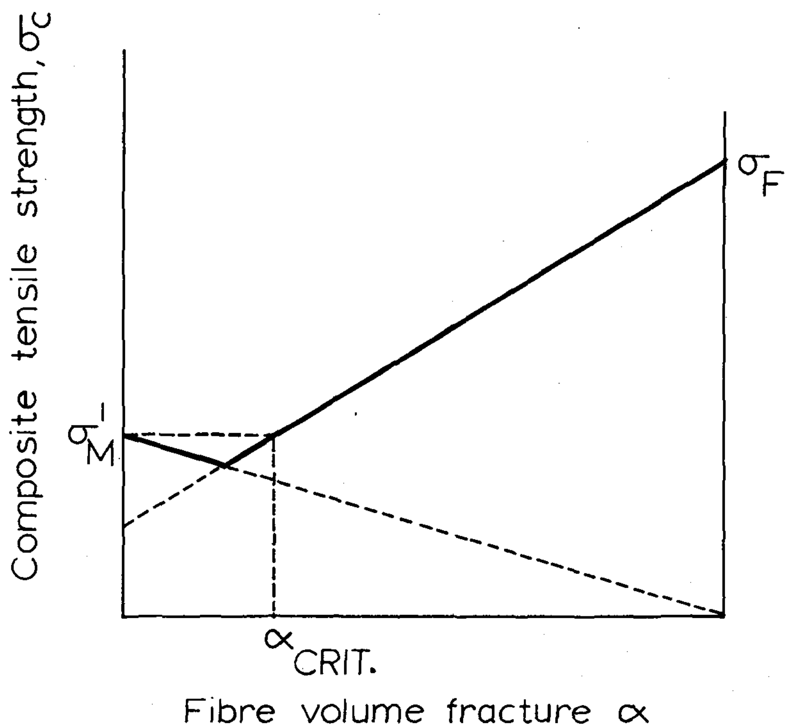


FIGURE 33 WEAKENING EFFECT AT SMALL VOLUME LOADINGS

approach of treating the fiber and matrix as a laminate sandwich (Figure 34). A given applied stress will result in additive strains so that:

$$1/E_t = \alpha/E_f + (1 - \alpha)/E_m \qquad \text{(Equation 4-7)}$$

If there is shear parallel to the sandwich, the strains will again be additive, so that:

$$1/G_{\ell t} = \alpha/G_f + (1 - \alpha)/G_m \qquad \text{(Equation 4-8)}$$

where

Subscripts t and ℓ stand for transverse and longitudinal directions, respectively

G_m denotes the shear modulus.

Where there is more than an order of magnitude difference between fiber and matrix stiffness, E_f and G_f may be regarded as infinite, so that the composite stiffness in these directions is approximately increased by a factor $(1-\alpha)^{-1}$ over that of matrix. If we assume the fiber modulus is 100 times the matrix (e.g., concrete) shear modulus and assume that 50 percent of parallel fibers in the matrix, Young's and shear moduli of the composite will be in the ratio 25:1. This difference between shear modulus and Young's longitudinal modulus in a composite also leads to an apparent lack of stiffness in the bending of short beams. This is why it becomes necessary to use a span-depth ratio of 40:1 for isotropic material (Tranopolisk, II et al., 1965).

Having discussed the basics, the typical load-deflection behavior in flexure is now considered. The load-deformation curve of a fiber-concrete specimen can be considered linear up to a point (see Figure 35) called "proportional or elastic limit" or "first crack strength"; beyond that point, the behavior can be termed "post-elastic limit" or "post-cracking" behavior. As stated previously, the origin of fibrous concrete being based on crack arrest and linear fracture mechanics concepts, the spacing or fiber reinforcement was thought of as the prime factor in predicting the elastic limit. However, the newer concept -- two-phase composite material -- is now accepted that relates the elastic limit to the volume, orientation and aspect ratio of the fibers.

If during the application of load the matrix cracks, the load is then entirely carried by the fiber; the point at which this starts is the "elastic limit" of the composite. Under flexural loads, a fiber reinforced beam, like any reinforced beam, cracks at a certain critical strain in outer fiber. However, the cracks do not propagate but are arrested by the fibers that span the cracks (Figure 36). Unlike an ordinary crack where no load can be maintained across it, in the fibrous concrete, some load is maintained across the crack due to the fiber bonded

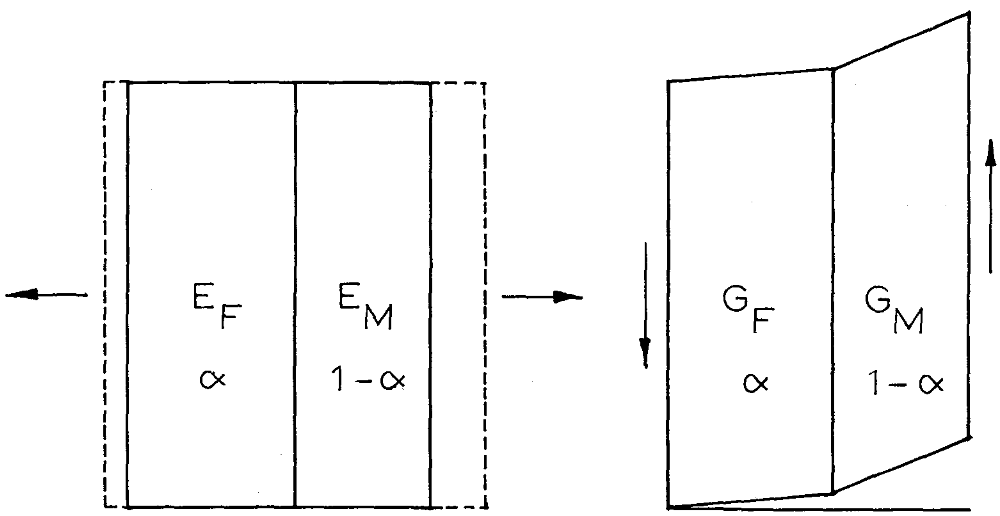


FIGURE 34 TRANSVERSE AND SHEAR STIFFNESS

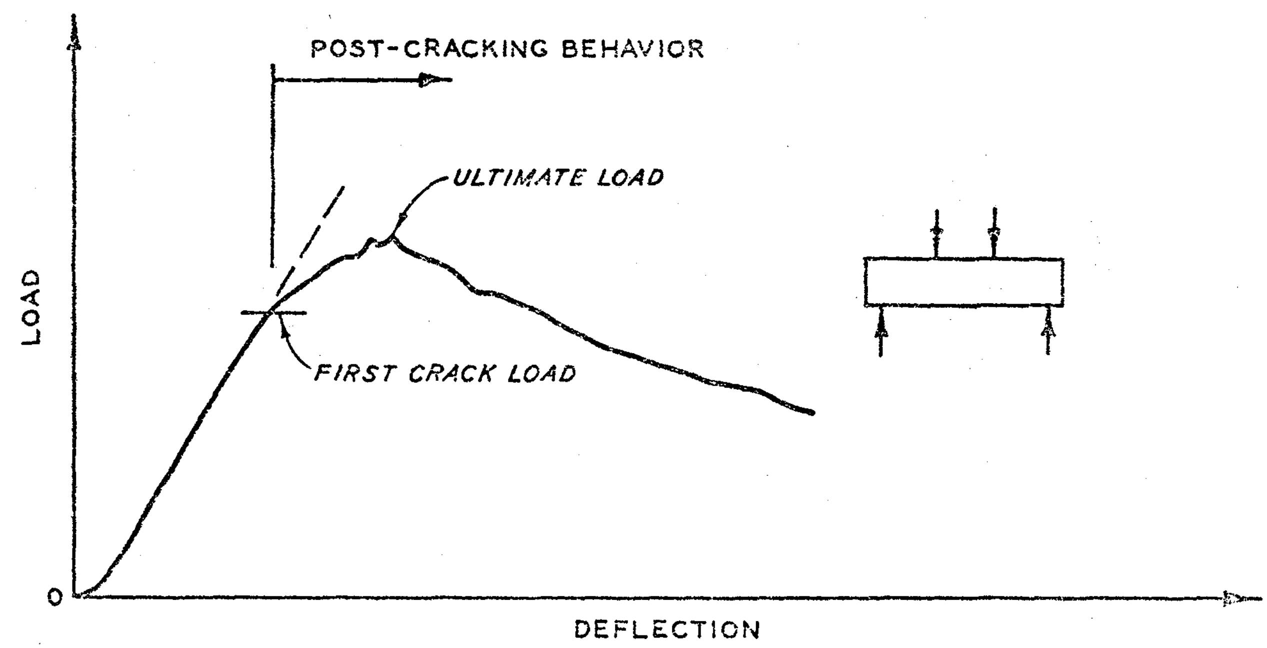


FIGURE 35 TYPICAL FLEXURAL LOAD-DEFLECTION BEHAVIOR OF STEEL FIBER CONCRETE

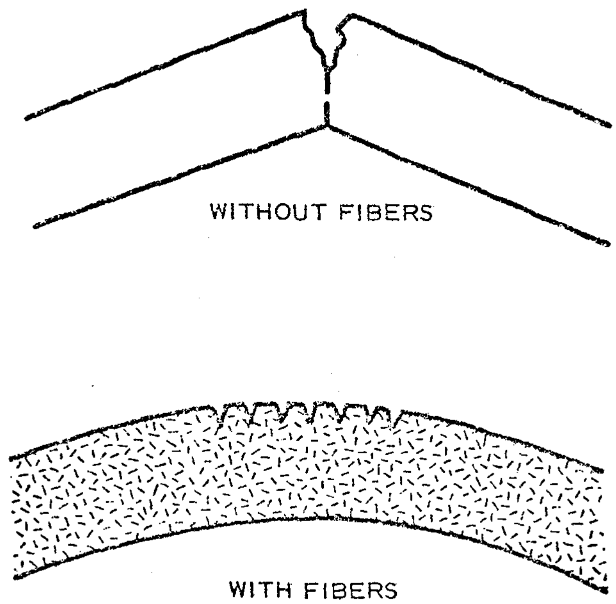


FIGURE 36 IDEALIZED CRACK ARREST MECHANISM IN BENDING

between the matrix. This load is dependent upon the embedment length of fiber, bond strength of fiber and matrix, strength of matrix and to a lesser extent on the strength of fiber itself and level of loading. With an increasing load, tensile strains increase and the location of the neutral axis continues to move up to a new location. After the cracking of the matrix, the failure of the fiber-reinforced concrete is generally attributed to the failure of the bond between fibers and matrix -- because the stress in the fiber is much less than the yield stress of fiber even up to the maximum load. It has been observed that certain chemical and mechanical treatments can substantially improve the bond strength as measured by a pull-out test on a single fiber (Tattersall and Urbanowicz, 1974); however, when such treated fibers are used in concrete, the observed improvement in the composite properties is much less significant than expected from the pull-out tests (Edington and Hannout, 1974). This indicates that perhaps the pull-out load of a single fiber does not provide an exact measure of the pull-out load when a group of fibers are pulling out from a cracked surface.

Naaman and Shah (1976) conducted a study to evaluate the pull-out behavior of the following three parameters: 1) angle of inclination of fibers with the loading direction; 2) number of fibers being pulled out simultaneously from the same area; and 3) the efficiency of random orientation. The results are described below:

1. The peak pull-out load for fibers inclined with respect to the loading direction is almost as high as those for fibers parallel to the loading direction (Figure 37). Since the peak pull-out load is considered a measure of bond efficiency, the results show that the bond efficiency of inclined fibers is essentially the same or better than that of parallel fibers;

2. Just prior to the complete pull-out, the load termed as final load (Figure 37) is zero for parallel fibers. The final load for inclined fibers increases with the angle of inclination and can be as high as the corresponding peak load;

3. The final pull-out distance is equal to the embedment for parallel fibers while it may be less than the embedment length for the inclined fibers (Table 13). The final pull-out distance is defined as the total amount of observed distance at which the pull-out load drops to zero;

4. The final load for inclined fiber increases with the diameter of fibers (Table 13, Figure 37);

5. The work required to completely pull out an inclined fiber is higher than that of a parallel fiber.

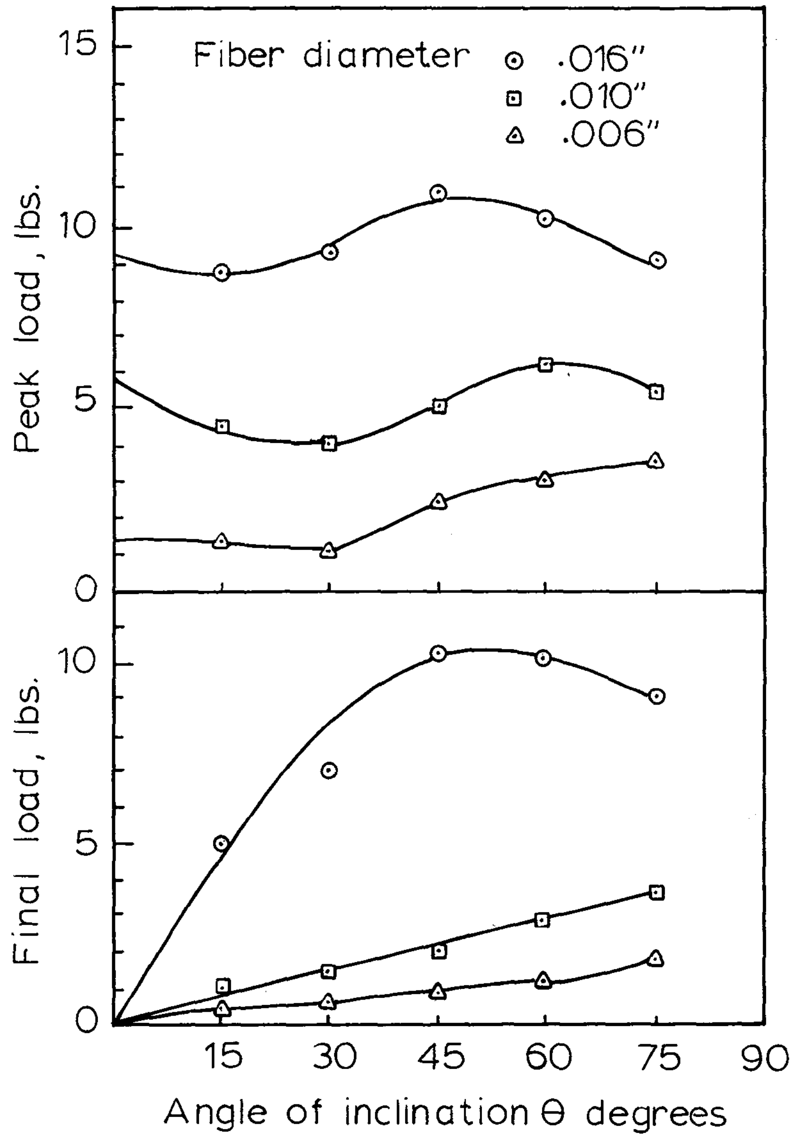


FIGURE 37 VARIATION OF PEAK AND LOADS WITH ANGLE OF FIBER ORIENTATION FOR DIFFERENT FIBER DIAMETERS (1 in.=25.4mm 1 lb.=4.45N)

TABLE 13

PULL-OUT TEST RESULTS AS FUNCTION OF NUMBER OF FIBERS

FIBER DIAMETER IN MILLIMETERS (1)	ANGLE OF ORIENTATION IN DEGREES, O (2)	DENSITY OF FIBERS PER SQUARE CENTIMETER (3)	PEAK LOAD PER FIBER, IN NEWTONS (4)	FINAL LOAD PER FIBER, IN NEWTONS (5)	FINAL PULL-OUT DISTANCE IN MILLIMETERS (6)
0.4064 (Music wire)	Parallel, O = 0	12.90	42.50	0.00	12.70
		25.80	34.09	0.00	12.70
		58.05	32.93	0.00	12.70
		103.20	36.49	0.00	12.70
		232.20	36.49	0.00	12.70
		522.45	_[a]	_[a]	_[a]
	Inclined, O = 60	12.90	44.28	39.83	10.92
		25.80	36.71	36.71	10.92
		58.05	35.82	34.40	10.16
		103.20	31.15	18.87	8.89
		232.20	28.04	0.00	6.35

[a]Concrete failed before pull-out.

[b]This is for two fibers spaced 1/2 in. apart.

Note: 1 in. = 25.4 mm; 1 lb. = 4.45 N; 1 fiber/cm^2 = 6.45 fibers/sq. in.

Figures 38 and 39 show the most significant trend observed:

1. Increasing the number of fibers from 2 in.^{2} (12.9 cm^2) to 36 in.^{2} (232.3 cm^2) does not significantly influence the peak pull-out load and the final pull-out distance for fibers aligned parallel to the direction of loading;

2. For fibers inclined at 60^o, the peak pull-out load, the final load, the final pull-out distance, and the total pull-out work decreases with an increasing number of fibers (Figure 39).

Thus for a group of inclined fibers, the contribution per fiber is less than that of a single fiber. This is analogous to a group of piles (deep foundation) wherein the bearing capacity of a group of piles is less than the sum of the individual piles.

Flexural Tests - Table 14 and Figure 40 show the results of flexural tests on various specimens. The efficiency ratio, defined as the ratio of modulus of rupture of the randomly oriented fibers to that of parallel fibers, is shown on Figure 40. The efficiency ratio decreases as the volume fraction of fibers increases. As the number of fibers increases, the contribution of randomly oriented fibers relative to that of parallel fibers decreases -- confirming the results of Figure 38.

The efficiency ratio observed in these investigations (0.66) is higher than many theoretical deductions (for example: Drenchal, 1964 - 3/8; Christensen, 1972 - 1/3; Abolitz, 1964 - 1/2; Cox, 1952 - 1/3). Laws (1971) arrived at similar values of efficiency ratios but with different considerations. Argon and Shack (1975) predict that efficiency ratios after cracking may have a value ranging from 0.4 to 1.0.

4.2.3 Fibers and Matrix Materials

Fibers of many types, sizes and shapes have been used but the most common are steel, glass, plastics and asbestos. Uncommon but in existence are fibers of aluminum alloys, tungsten wire and carbon fibers. The natural packing of rigid fibers depends on their aspect ratio, defined as the length divided by an equivalent diameter of fiber. For three-dimensionally random fibers, volume concentration in space is proportioned to $(L/d)^{-2}$. When fibers are random in plane, packing is proportioned to $(L/d)^{-1}$, whereas the packing of truly parallel fibers is independent of aspect ratio: typical aspect ratios range from about 30 to 150 for fiber length dimensions of 0.25 to 3 in (6.4 to 76.2 mm). Table 15 provides typical properties of fibers.

Steel fibers in concrete have been extensively studied, their use originating from tunnel linings and extended to pavement applications. Enormous literature and state-of-the-art

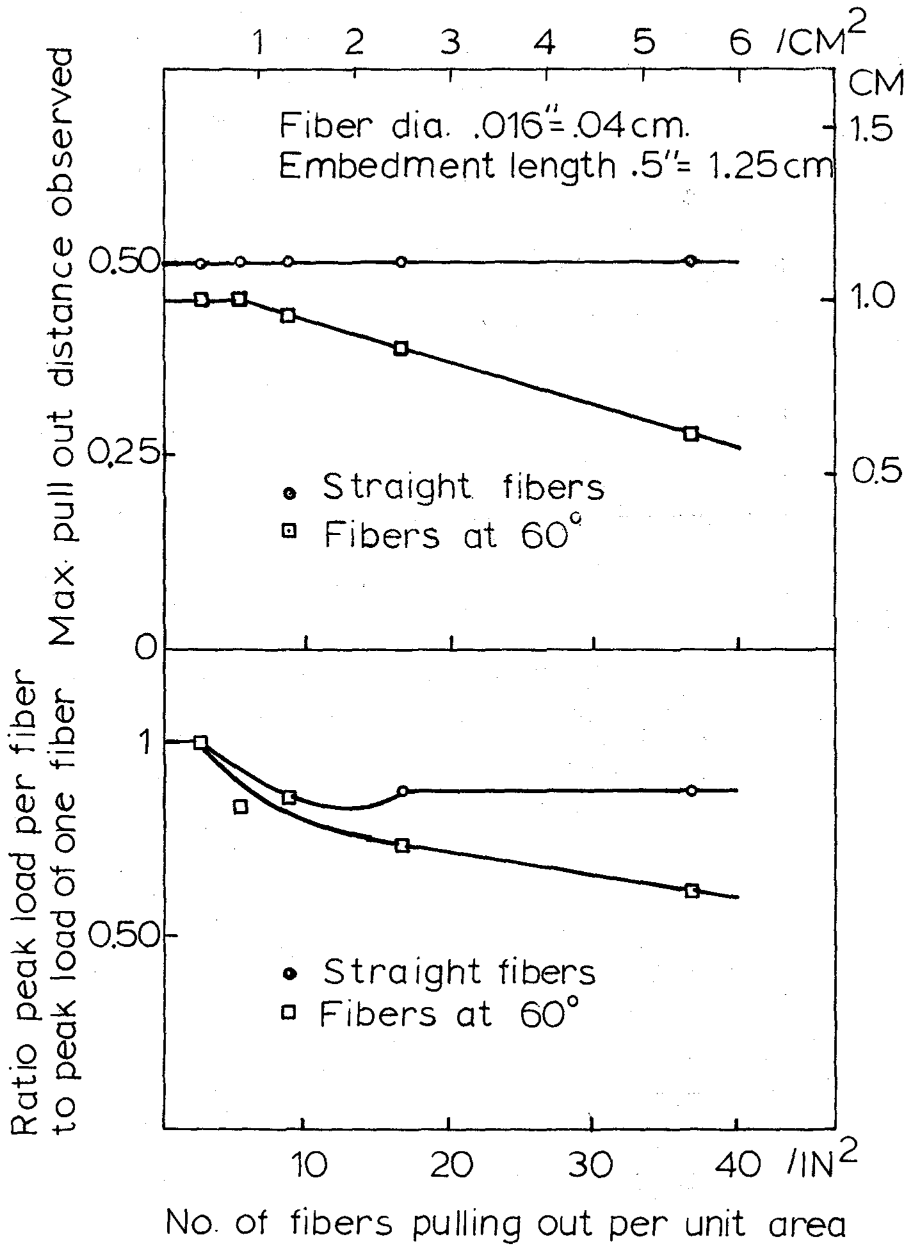


FIGURE 38 INFLUENCE OF NUMBER OF FIBERS ON PULL-OUT WORK FOR ALINED AND INCLINED FIBERS (1 in.=25.4mm)

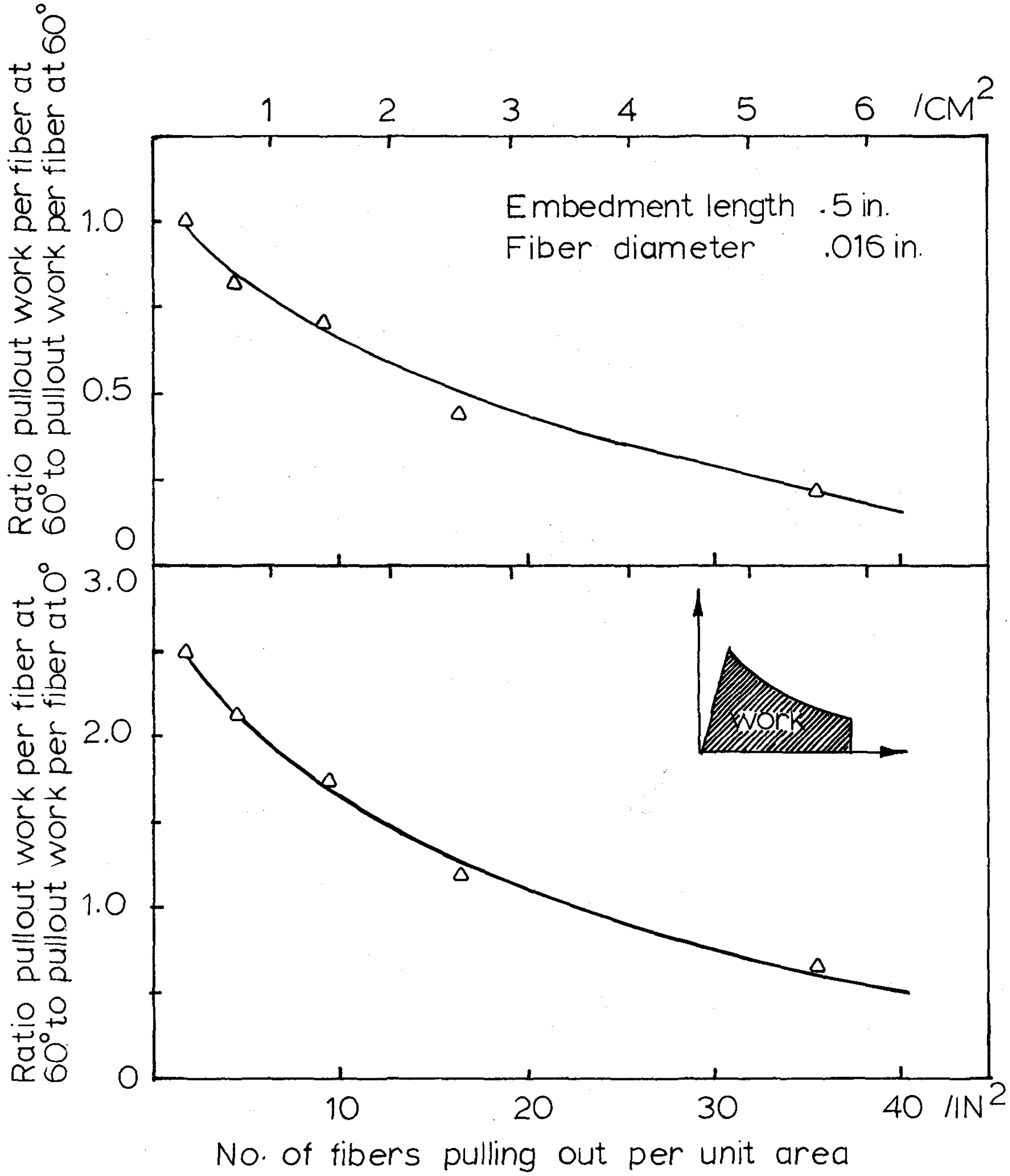


FIGURE 39 INFLUENCE OF NUMBER OF FIBERS ON PULL-OUT LOAD AND DISTANCE FOR ALINED AND INCLINED FIBERS (1 in.=25.4mm)

TABLE 14

RESULTS OF FLEXURAL TESTS WITH PARALLEL AND RANDOMLY ORIENTED FIBERS

VOLUME FRACTION AS A PERCENTAGE (1)	FIBER DIAMETER 0.4064 mm, FIBER LENGTH 25.4 mm			FIBER DIAMETER 0.254 mm, FIBER LENGTH 25.4 mm		
	PARALLEL (2)	RANDOM (3)	EFFICIENCY RATIO (4)	PARALLEL (5)	RANDOM (6)	EFFICIENCY RATIO (7)
Modulus of Rupture in Kilonewtons per square meter*						
0.5	5,548	5,051	6.28	6,044	5,216	61.13
1.0	7,204	5,879	5.66	8,032	5,548	4.76
2.0	9,936	7,121	4.97	1,587	10,516	4.55
First Crack Stress in Kilonewtons per square meter*						
0.5	4,802	5,051		4,471	5,134	
1.0	5,796	5,713		5,051	5,382	
2.0	6,548	5,216		6,293	6,458	

*1 in. = 25.4 mm; 1 psi = 6.9 kN/M^2.

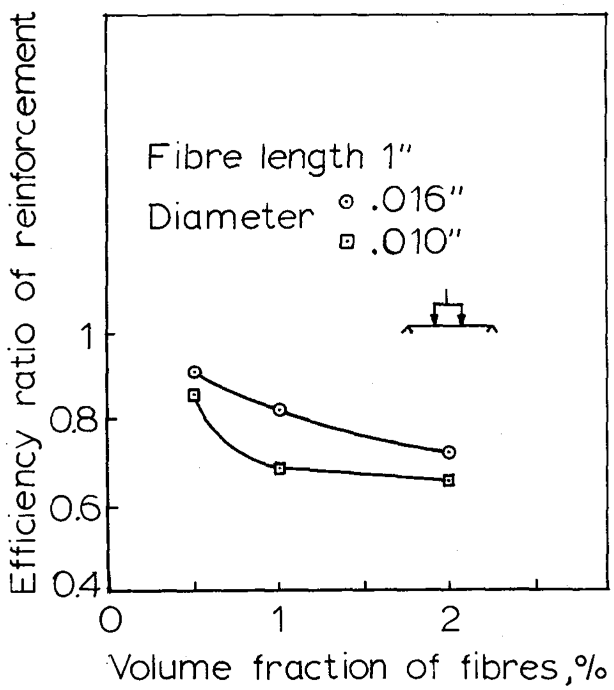


FIGURE 40 OBSERVED VARIATIONS OF EFFICIENCY RATIO OF FIBER ORIENTATIONS IN FLEXURED PLATES

(1 in = 25.4 mm)

TABLE 15

TYPICAL PROPERTIES OF FIBERS

TYPE OF FIBER	TYPICAL DIAMETER in. X 10^{-3}	MAXIMUM FIBER LENGTH (in.)	SPECIFIC GRAVITY	TENSILE STRENGTH psi X 10^3	ELASTIC MODULUS psi X 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						
Chrysolite (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	2½	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 (Scandinavian)	0.08-0.2	2	2.7	70-110	10-17	≈0.6
Alumino-Silicate	0.11	10	2.7	-	-	-

1 in = 25.4 mm   1 psi = 6.9 kN/m^2

reports are available on steel fibers. Round steel fibers in use in the United States typically have a diameter ranging from 0.006 in. to 0.025 in. (0.152 to 0.635 mm) and 0.75 in. (19.05 mm) in length. Flat steel fibers have dimensions typically 0.010 in. by 0.222 in. by 1 in. (0.254 by 5.64 by 25.4 mm) long. Longer fibers than 1 in. (25.4 mm) are being used in the United States. Crimped and deformed steel fibers have been produced to improve bond. For the same reason, numerous coatings have been applied and are in use. Table 16 shows the effect of chemical and mechanical treatments on the pull force.

Glass fibers have diameters of 0.2 mils to 0.6 mils (5.1 to 15.2 microns), but these fibers may be bonded to produce glass fiber elements with diameters of 0.5 to 50 mils (12.7 to 1270 microns). The soda-lime-silica glass in literature is referred to as A-glass and bore-silicate glass as E-glass. A glass containing zirconia (ZYO_2) is now available and is called AR-glass.

Polymeric fibers of nylon, polypropylene, polyester and rayon are available, but for pavements their contribution is not significant. Fibers processed from natural materials like asbestos, cotton and vegetables provide a wide range of sizes.

New fundamental research has shown that at very high temperatures (2000°C) graphite could be oriented or drawn by an applied tension. Thus, it is possible to convert the random microcrystalline structure of rayon-based carbon fiber to an oriented structure by drawing it at a very high temperature. This led to the small-scale production in the United States (Bacon, 1969) of a family of carbon fibers of high modulus in the range 25 to 60 x 10^6 psi (1.76 to 4.2 x 10^6 kg/cm^2) as continuous. Much effort has and is going on into the development of good carbon fiber. Only one proprietary organic fiber, DuPont PRD 49, with a high strength of 500 ksi at 200°C has been reported. Data on this product are very limited.

Matrix materials are portland cement, high aluminum cement and gypsum plaster, similar to concrete. Fibrous concrete requires a considerably greater amount of fine material in the mix than does plain concrete for it to be conveniently handled and placed by current procedures and equipment. It has also been shown that cement content can be significantly reduced and quality maintained; if fly ash is substituted for a portion of cement, a water reducer is used. The concrete is air entrained and the fineness modulus of aggregate is increased (Kesler, 1972). Such a mix is easier to place. Significant gains occur in such a mixture due to the pozzolanic nature of fly ash.

To achieve a durable fibrous concrete, some requirement on the concrete constituents can be demanded. The hardened cement paste can be considered a continuous constituent while the fiber and aggregate are dispersed or discontinuous constituents. The

TABLE 16

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[b]
Electrogalvanized (28-day)	6.3	Imitation Duoform[a]	>10.4[b]
Hot-dip galvanized + furnace	6.1	Solder blob (28-day)	> 9.3[b]
Hot-dip galvanized (28-day)	6.0	Duoform[a]	> 9.2[b]
Rusted (28-day)	5.8	Duoform[a] (28-day)	> 8.8[b]
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 + chromate	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

[a]Trademark of National-Standard Company, patented shape.

[b]Reference 242, data from G.H. Tattersall and C.R. Urbanawicz (University of Sheffield), pp. 17.

TABLE 16 (continued)

CHEMICAL TREATMENTS	BOND IMPROVEMENT FACTOR	MECHANICAL TREATMENTS	BOND IMPROVEMENT FACTOR
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		
Oxidized 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		

cement matrix normally contains pores, voids and micro cracks that result in a poor bond to the fibers and can allow the intrusion of aggressive fluids into the concrete. Since the matrix has low tensile strength, the matrix cracks when fibers begin to reach their full load carrying capacity. To satisfy the zero-maintenance criteria, the matrix should be dense and impermeable, have low absorption and shrinkage. This can be achieved partly by using the lowest possible water-cement ratio and proper consolidation.

The normal hydration of portland cement results in the liberation of calcium hydroxide, which forms a saturated solution in water of very high pH (12 to 14). This solution resulting from the presence of alkali hydroxides collects in the pore space of the cement gel and tends to protect steel but has a degrading effect on glass.

In summary, one can say that the following factors affect the strength in addition to the fiber variables (i.e., length, diameter, shape and percentage by volume of fibers):

- The age of the specimen at testing
- The water-cement ratio of the matrix mix
- The cement factor of the mix
- The aggregate size and gradation
- The cement source
- The uniformity of fiber dispersion
- The consolidation techniques
- Bond strength
- The surface condition of the fiber

It may also be noted that specimen size and type of test -- tensile or flexural -- also usually affect the results.

4.2.4 Engineering Properties

4.2.4.1 Steel Fibrous Concrete

Steel fiber improves the engineering properties of concrete. An infinite number of combinations of fiber variables (i.e., length, diameter, shape and percentage by volume) are possible within the limits set for conventional mixing operations. It is also agreed that with higher volumes of fiber or with higher aspect ratios, higher strengths are possible, but workability is reduced. A choice between strength and workability becomes a controlling factor.

Static Strength - Theoretically, the first-crack strength (elastic limit) should not be influenced by the addition of fibers; however, the results show that with steel fibers, up to 4 percent by volume, the first-crack strength increased as much as 2.5 times the strength of plain concrete. The ultimate strength depends on volume, aspect ratio, specific surface, embedment length and the cement content of concrete. If regulated-set cement is used instead of ordinary cement and mix proportions are kept the same, the flexural strength and first-crack strengths of the concrete with regulated-set cement were found to be higher. The tensile strength of fibrous steel concrete increases with increasing fiber content as do the tensile strains. However, an increased aspect ratio improves the behavior. If the fibers are randomly distributed, the tensile strength is about 90 percent of the case when fibers are aligned parallel to the direction of stress. Splitting tensile strength of mortar reinforced with steel fibers was reported to be about 2 to 5 times that of unreinforced mortar when 3 percent fiber by volume was used and 2 times when 1.5 percent was used.

The mechanism of shear failure in compression is complex and not well understood. The reported compressive strengths of unidirectional composites may be high, under ideal conditions, but these values may not be correct for normal means of construction and loading. Design for high working stresses is inadvisable unless reproducibility can be thoroughly tested on actual pavements. With 2 to 40 percent steel fibers by volume, the compressive strength has been reported to be from 10 to 50 percent more than the strength of plain concrete.

Modulus of Elasticity - The modulus of elasticity of steel fibrous concrete is basically the same as that of a concrete without steel fibers, ingredients of concrete being the same. If cement content is increased, the modulus of elasticity also increases. The modulus of resilience, on the other hand, is influenced by both the fiber content and the cement content of the mix. An increase of either would result in a higher modulus of resilience.

Ductility and Creep - Up to the elastic limit, the ductility of steel fibrous concrete is the same as that of a plain cement concrete. However, beyond the elastic range, the presence of a hard particulate leads to an increased work-hardening and to reduction of elongation to fracture. If the fibers used are below their critical aspect ratio, the ductility in tension is increased. In such a case, if the matrix adheres to the fiber, the observed work-hardening rate is very high. The mechanism is the same by which short fiber reinforcement reduces axial creep (i.e., plastic flow over a period of time at a given temperature and stress).

Toughness - The presence of steel fibers in plain concrete increases the toughness of concrete (even though ultimate load may be the same). So the steel fibers are directly responsible for increase in toughness and improving the resistance to crack propagation.

Dynamic and Fatigue Strength - Dynamic and explosive load tests (Birkmer et al., 1968; Williamson, 1966) demonstrated that steel fibrous concrete has an increased resistance to spalling and has increased impact and dynamic strength. Steel fibrous concrete exhibited better dynamic strength than concrete made with plastic fibers. The greater energy requirements to pull out the fibers provides impact strength and resistance to spalling and fragmentation.

As for fatigue behavior, fatigue strength is increased due to use of a high modulus fiber (even though in general, fracture energy has incidentally fallen or remained constant). Several experimental fatigue studies have been conducted; however, beam sizes, loading conditions and fatigue strengths of from 90 percent at 3×10^6 cycles to 50 percent at 10×10^6 cycles for non-reversal-type loading using 2 to 3 percent steel fiber by volume. Reversal-type loading indicated fatigue strengths of 73 percent for 2 to 3 percent steel fiber by volume (ACI Committee, 1973). Post-fatigue static flexural strength was 10 to 30 percent greater than for similar beams with no fatigue loading history. One explanation provided is that the cyclic loading reduced initial residual tensile stresses due to shrinkage of the matrix by accelerating the creep. However, for fiber contents of 2 percent or less, which can be mixed and placed in the field, the results are not as conclusive.

Environmental Effects - Little data exist on the behavior of fibrous steel concrete under adverse environments. However, it has been reported that the inclusion of steel fibers (0.5 to 1.5 percent by volume) did not influence the temperature gradient. Similarly, the drying shrinkage and thermal expansion properties of fiber-reinforced concrete were found to be similar to those of plain concrete. Unpublished data for 2 percent by volume fiber-reinforced mortar tested in a vacuum condition showed a 30 percent increase in thermal conductivity as compared with plain concrete when exposed to a temperature range of from $-20^{\circ}C$ to $120^{\circ}C$ (Schwarz, 1972). Electrical conductivity of steel fibrous concrete is reported to be slightly higher than plain concrete in tests involving passing current between fittings in plain and fibrous concrete rail cross ties. No data are available on the permeability of fiber-reinforced concrete in general. Intuitively however, it would seem that for a mix of similar ingredients, the permeability of plain cement concrete and fiber-reinforced concrete should be the same.

The durability of steel fibrous concrete is dependent on the rusting of steel fibers. In cracked steel, fiber-reinforced concrete does not deteriorate when exposed to a variety of exposure conditions (Edgington et al., 1975), but the cracked fibrous concrete will lose its effectiveness with prolonged exposure, due to rusting of exposed fibers. A parametric study, performed at the Illinois Institute of Technology (Appendix II), of the effects of lost fiber capacity indicates that the loss of fiber capacity in a modulus of rupture of beam results in loss of beam toughness rather than strength. The loss of capacity of the fibers in a beam on an elastic foundation results in lower fiber concrete stresses but larger deflections and soil stresses near the load. The tensile stresses in the fiber concrete near a cracked section are influenced by the capacity of the fibers to transmit forces across a cracked section.

The addition of steel fibers to concrete does not result in improved freeze-thaw resistance over the same mix of ordinary cement concrete.

Abrasion and erosion of steel fibrous concrete surfaces and plain concrete surfaces have been simulated by sand blasting and electrically driven rotary steel brushes. Based on visual examination, fiber concrete surfaces exhibit less spread and depth deterioration with increasing fiber content. With simulated abrasion and erosion of the surface, the fibrous concrete had up to 15 percent higher skid and rolling resistance than plain concrete under dry, wet, and frozen surface conditions. In summary, the properties of a steel fibrous concrete are dependent on length, diameter, shape and percentage by volume of the fibers. Higher volumes of fiber or fibers with higher aspect ratios may provide high strengths but fibers more than 2 percent by volume decrease workability. The following ranges of improvement in the properties reported below relate to fiber contents of from 0.25 to 2 percent by volume (33 to 265 pounds per cubic yard or approximately 20 to 160 kg/m^3)(Schwarz, 1972).

PROPERTIES	APPROXIMATE MAXIMUM IMPROVEMENT OVER PLAIN (NONREINFORCED) CONCRETE IN PERCENT
Compression	up to 30
Flexural Modulus of Rupture	up to 200
Tensile Strength (Direct & Indirect)	up to 200
Impact Strength and Spall Resistance	up to 1000
Crack Resistance	up to 200
Fatigue Strength to 2 million cycles	up to 50
Abrasion	up to 30
Shear and Tensional Strength	up to 300
Corrosion Resistance	As good as plain concrete
Freeze-Thaw	As good as plain concrete

4.2.4.2 Reinforced Fibrous Concrete

Not much work has been done on beams or slabs reinforced with steel bars in addition to steel fibers. Henager and Doherty (1976) developed an analytical method based on an ultimate strength approach to calculate the flexural strengths of reinforced steel fibrous concrete beams. The method takes into account the bond stress, fiber stress, fiber length/diameter ratio and volume of the fiber. The strength computed for the fibrous concrete is added to the strength contributed by the reinforcing bars to obtain the theoretical ultimate moment. The method showed a good relation between predicted strength and experimental values. The following experimental results are noteworthy:

1. The use of steel fibrous concrete in reinforced concrete beams has been shown to increase their ultimate flexural strength. Strength increases were on the order of 25 percent for the particular beams tested;

2. Crack width and crack spacings were less in reinforced steel fibrous concrete beams, and the first cracks occurred at higher loads;

3. The post-cracking stiffness of the reinforced fibrous beams was greater than of a conventional reinforced beam.

4.2.4.3 Glass Fibrous Concrete

Though the first crack, flexural strength, ultimate flexural strengths and the impact strength of glass fibrous concrete have been found to increase linearly with increased volume percentage, durability is a prime consideration for a material to be selected as a zero-maintenance pavement material. As pointed out in Section 4.2.3, boro-silicate E-glass fibers have a known history of deterioration in the alkali environment of a cement base (Blood, 1970; Ali and Grimer, 1969). New alkali resistant (AR) glass fiber containing zirconia (ZYO_2) is now available, which is far better than the E-glass. However, even this glass is not totally immune from reduction in strength when surrounded by a cementitious matrix. Available information is conflicting as to the exact cause of degradation and the degree of severity. The flexural strength of the composites made with AR-glass fibers was found to have an irregular progressive decrease beginning after 1 month and lasting about 5 to 6 months (Figure 41). Cohen and Diamond (1975) argue that the decrease in strength should not be attributed to alkaline attack on the fibers solely; however, they failed to provide another explaining mechanism for the decrease.

Dynamic fatigue life of glass fibrous concrete has been studied by Hibbert and Grimms (1974), and a fatigue life of 1 million cycles was recorded for specimens stored 1 year under various environmental conditions including natural weathering. Some

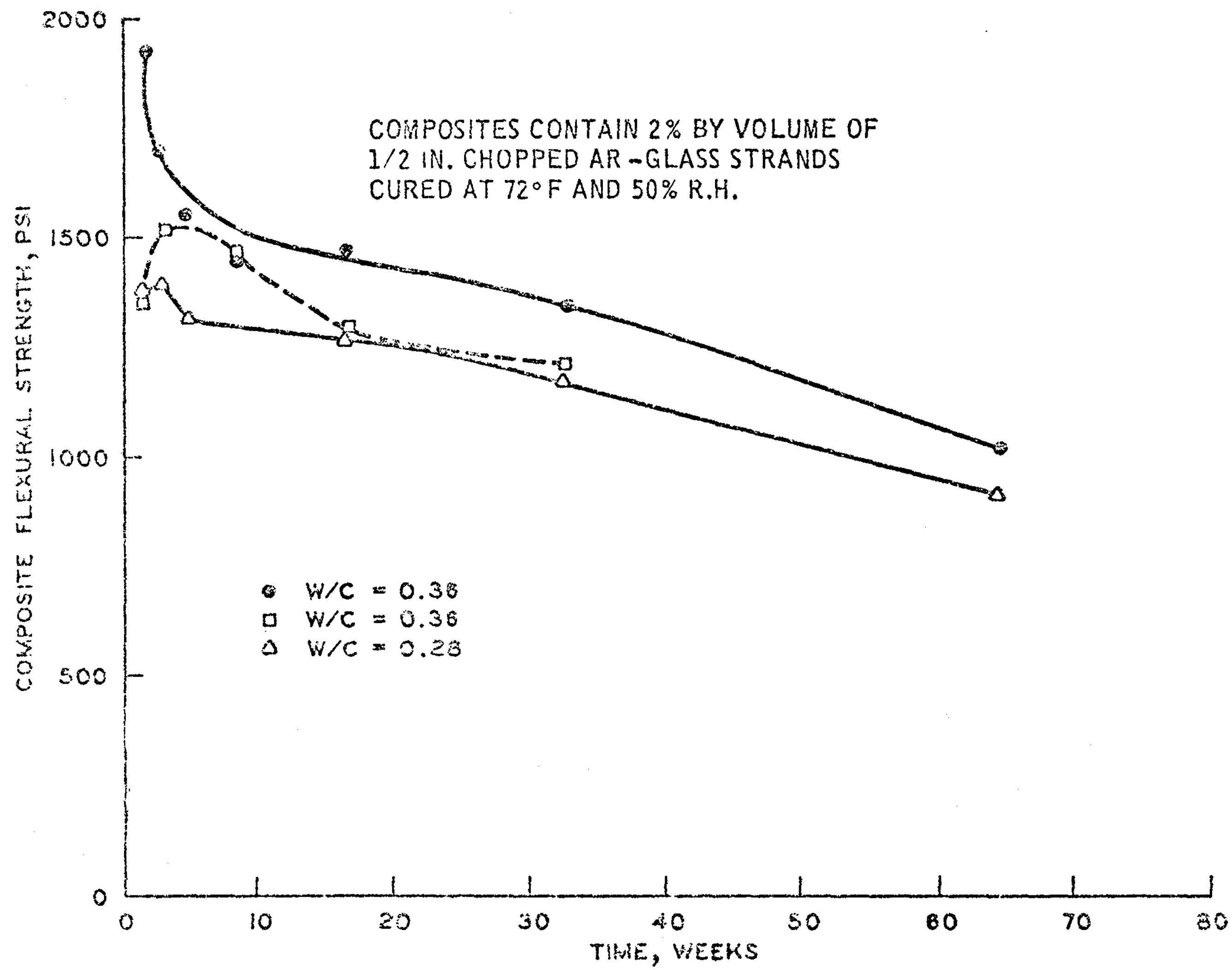


FIGURE 41 FLEXURAL STRENGTH VS. AGE RELATION FOR AR-GLASS REINFORCED CEMENT COMPOSITES

(1 psi = 6.9 kN/m^2)

of these results are shown on Figure 42. For a given stress, the fatigue life was found to increase with increasing fiber content, but at levels above the elastic limit, the fatigue life of glass fibrous concrete was found to reduce appreciably with magnitude depending on the applied stress.

Steele's (1972) findings indicate that the glass fibrous concrete degrades in impact resistance with time, regardless of whether it is cured in water or in air. This loss is shown on Figure 43 for both E- and A-glass fibers. Though this concrete (composite) loses impact strength, yet the residual strength is 15 times that of no-fiber matrix and 5 to 10 times that of asbestos reinforced cement.

The tensile and compressive strength of the glass fibrous cement are fairly comparable to steel fibrous concrete within the limits of workability. Nevertheless, the knowledge of the properties available suggests that the use of glass fibrous concrete in a zero-maintenance pavement is not warranted essentially on the basis of durability and fatigue life.

4.2.4.4 Polymeric Fibers

It has been observed that the bond of a cement matrix to a polymeric fiber is generally not good. Within a fiber volume fraction of less than 1 percent, the total load carried by the fibers is very small. After cracking, the stresses in the fiber are governed by bond and the rather poor bond causes the post-cracking strength to be fairly constant (as the fibers debond and pull out easily). Dardare (1975) found that some increase in flexural strength occurs in polypropylene fibrillated fibrous concrete, and this increase is dependent on both the length and volume fraction of the polypropylene fiber (Figure 44 and 45). It can be observed that when volume fraction exceeded 0.6 percent, the strengths decreased from the maximum, and for more than 1.0 percent fiber, strengths were less than the no-fiber mixture. Monfore (1968) reported progressive decreases in strength with nylon fiber volume for mortar.

A study of the tensile and compressive strengths of polymeric fibers indicates that the improvement of strength is not worth the trouble. Based on these facts, the investigators do not feel that polymeric fibrous concrete has at least an immediate future in zero-maintenance pavements.

4.2.4.5 Fibrous Concrete from Natural Materials

Fibers such as cotton, vegetable fibers (and rayon, acrylic and polyester) are subject to alkaline attack and are not very effective.

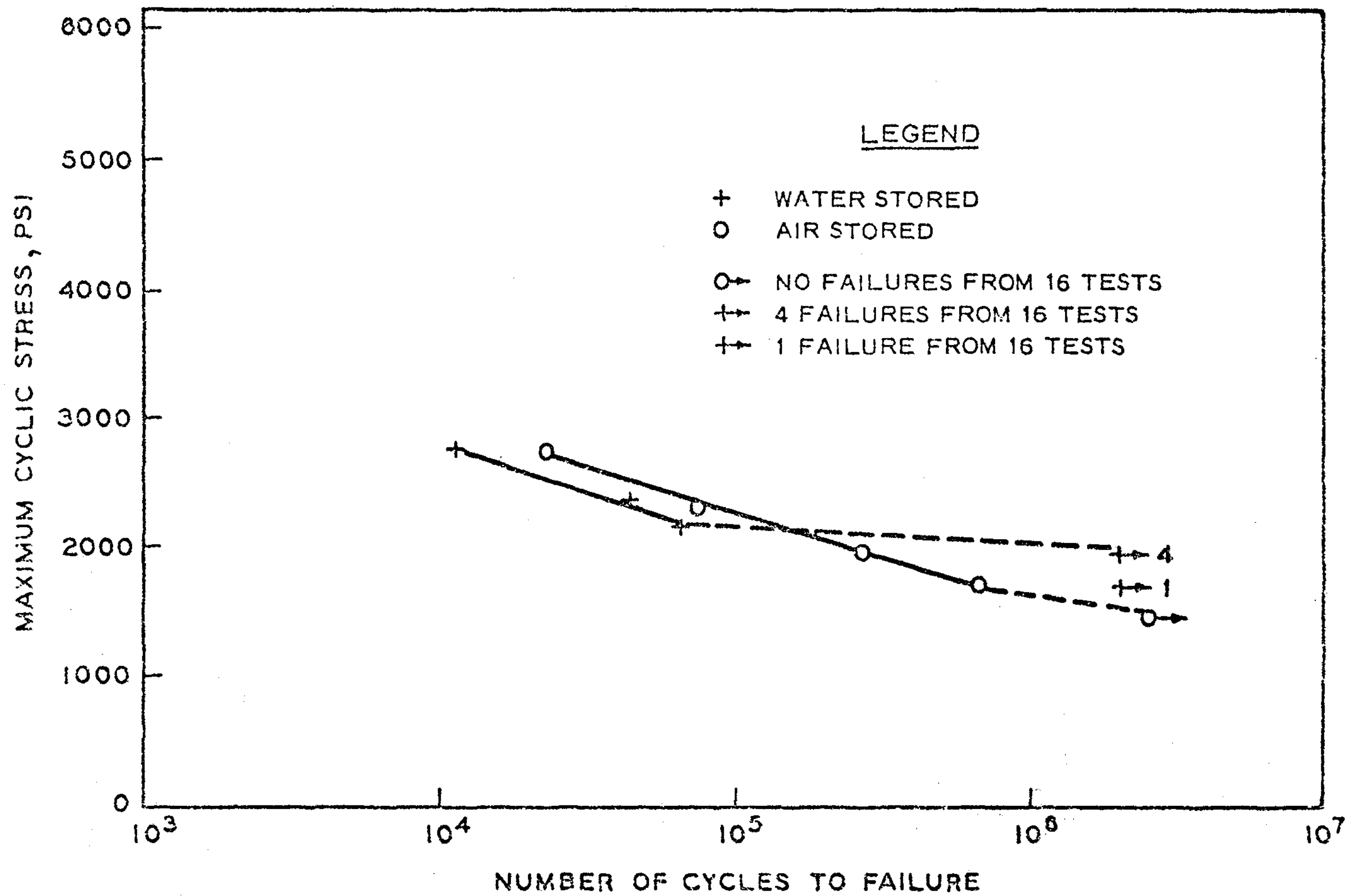


FIGURE 42 FATIGUE LIFE OF GLASS FIBER REINFORCED CEMENT

(1 psi = 6.9 kN/m^2)

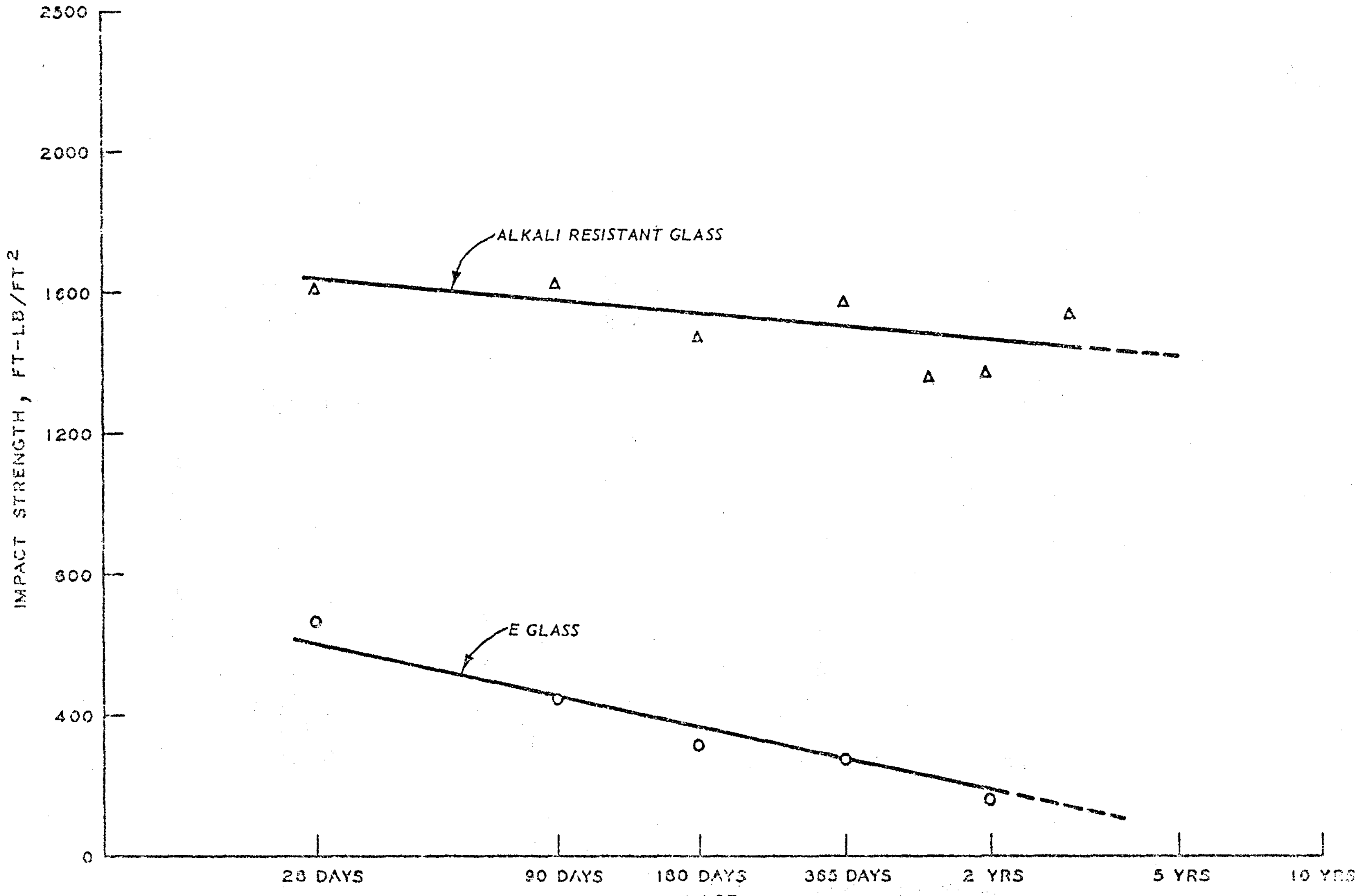


FIGURE 43 DURABILITY OF IMPACT RESISTANCE OF E-GLASS AND AR-GLASS FIBERS IN AN ORDINARY PORTLAND CEMENT MATRIX

(1 ft-lb/ft^2 = 14.5 m-N/m^2)

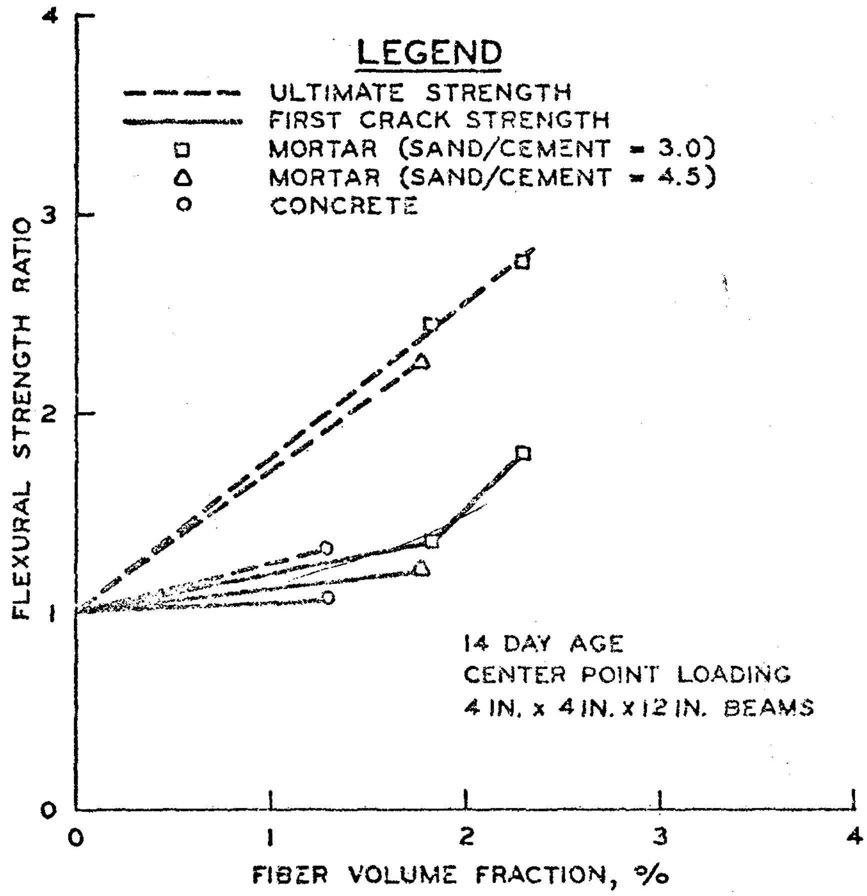


FIGURE 44 FLEXURAL STRENGTH RATIO VERSUS FIBER VOLUME FRACTION FOR 2/3/4 IN. FIBRILLATED POLYPROPYLENE FIBER REINFORCED MOTARS AND CONCRETE

(1 in = 25.4 mm)

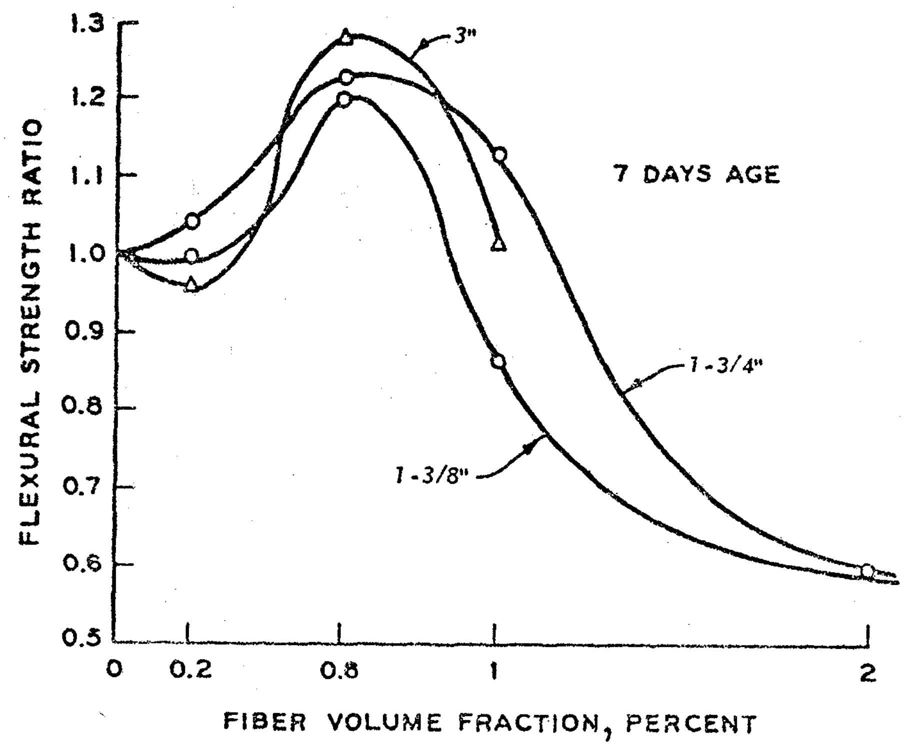


FIGURE 45 FLEXURAL STRENGTH RATIO VERSUS FIBER VOLUME FRACTION FOR VARIOUS LENGTHS OF FIBRILLATED POLYPROPYLENE FIBER REINFORCED CONCRETE

(1 in = 25.4 mm)

Asbestos fibers do provide increased flexural strength, but the increase in tensile strength is not very significant (Figures 46 and 47). The post-cracking strength (from limited available data) was as good as of plain concrete (i.e., nothing). Asbestos fibrous concrete will have significantly improved impact resistance as per studies of Majumdar (1970) and Ali et al. (1969). Not much data are available on asbestos fibrous concrete.

Carbon fibers have been and are being studied. A discussion on their engineering properties follows.

Using aligned carbon fibers, investigators have found a six-fold increase in flexural strength with increasing fiber volume up to 13 percent (Walter, 1974). Random chopped carbon fibers in spray-suction applications provided a modulus of rupture as high as 4000 psi (281 kg/cm^2). The tensile strength of about 14,000 to 15,000 psi (984 to 1055 kg/cm^2) with 9 to 10 percent carbon fibers (compared to 400 to 600 psi (28.1 to 42.2 kg/cm^2) of portland cement concrete) has been observed. However, a look at Figure 48 will reveal that under tensile strain, the amount of strain can be 10 to 40 times what might be expected for a plain cement paste.

No data on compressive strength of carbon fibrous concrete were available. Carbon fibers have been found to reduce flexural creep deflection by a factor of 6 at 2 percent volume of fibers (Briggs et al., 1974).

Carbon fibers appear to perform well in an environment of cement. Ali et al. (1972) and Briggs et al. (1974) found good durability and no change in modulus of rupture up to 1 year of exposure for water and air storage at 64°F (17.8°C).

Freeze and thaw resistance of carbon fibrous concrete was found to be good by Briggs et al. (1974). The same authors 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 (α = 7 percent) showed a consistent deterioration in strength while in the loaded condition; this loss ranged from 17 to 27 percent with respect to the unloaded condition. Dry air environment was found to be the most deteriorating irrespective of the level of stress.

Briggs et al. 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 on Figure 49. The tendency of

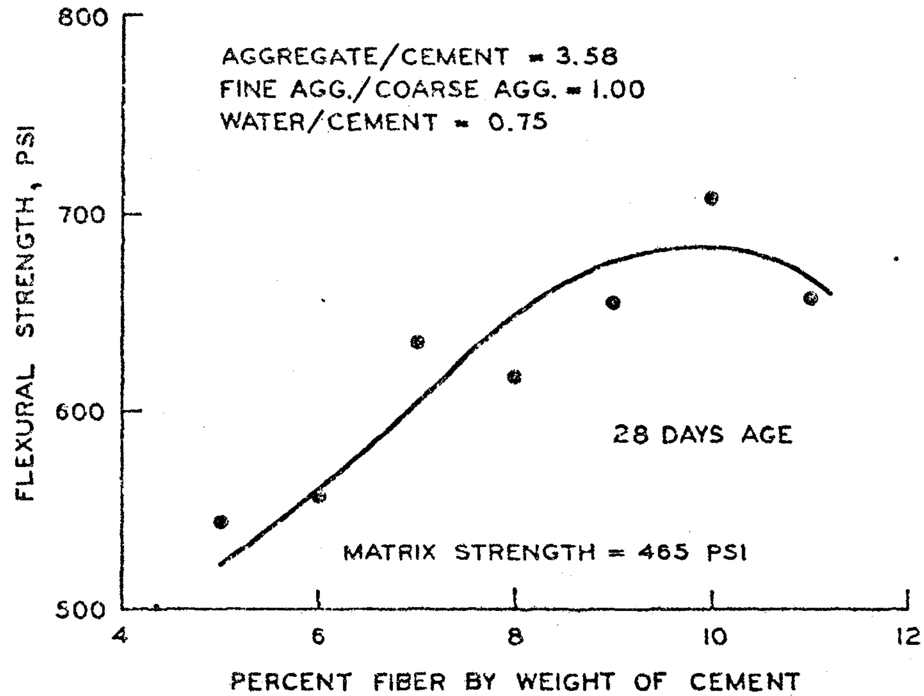


FIGURE 46 FLEXURAL STRENGTH VERSUS FIBER CONTENT RELATION FOR ASBESTOS FIBER REINFORCED CONCRETE

(psi = 6.9 kN/m^2)

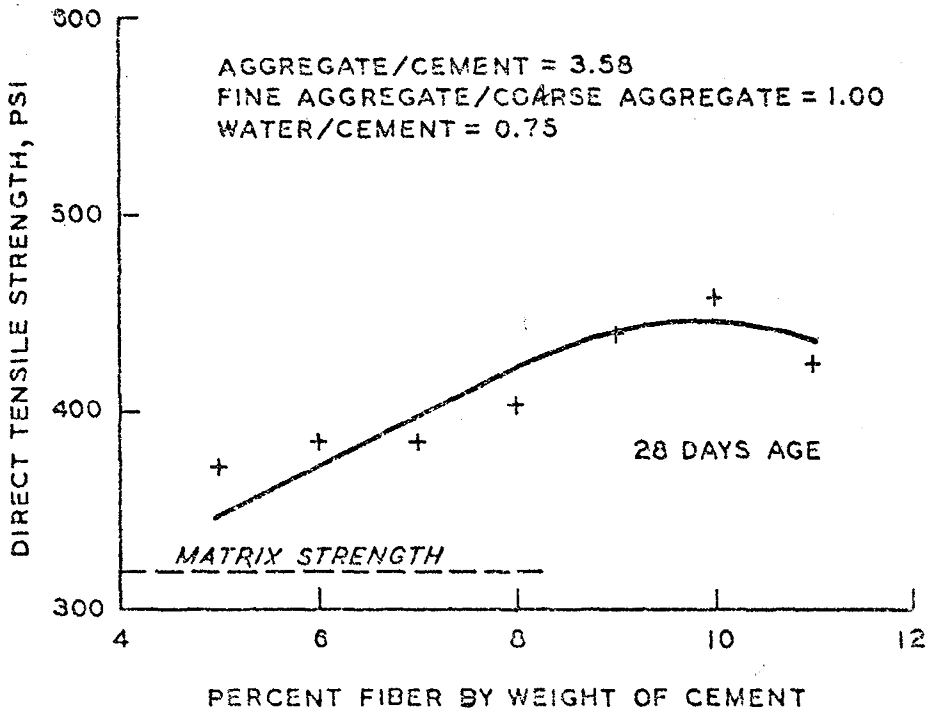


FIGURE 47 TENSILE STRESS VERSUS FIBER CONTENT RELATION FOR ASBESTOS FIBER REINFORCED CONCRETE

(1 psi = 6.9 kN/m^2)

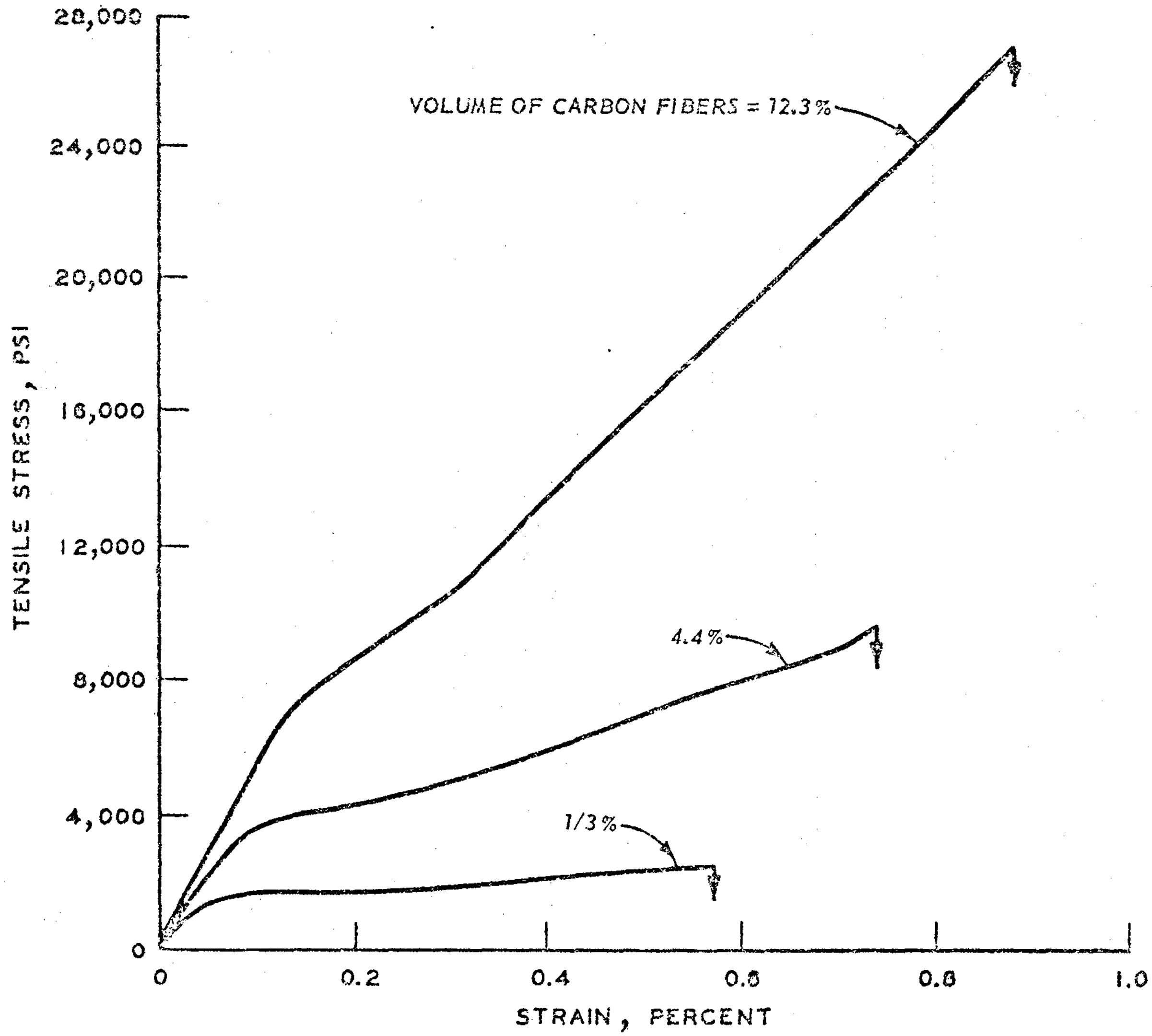
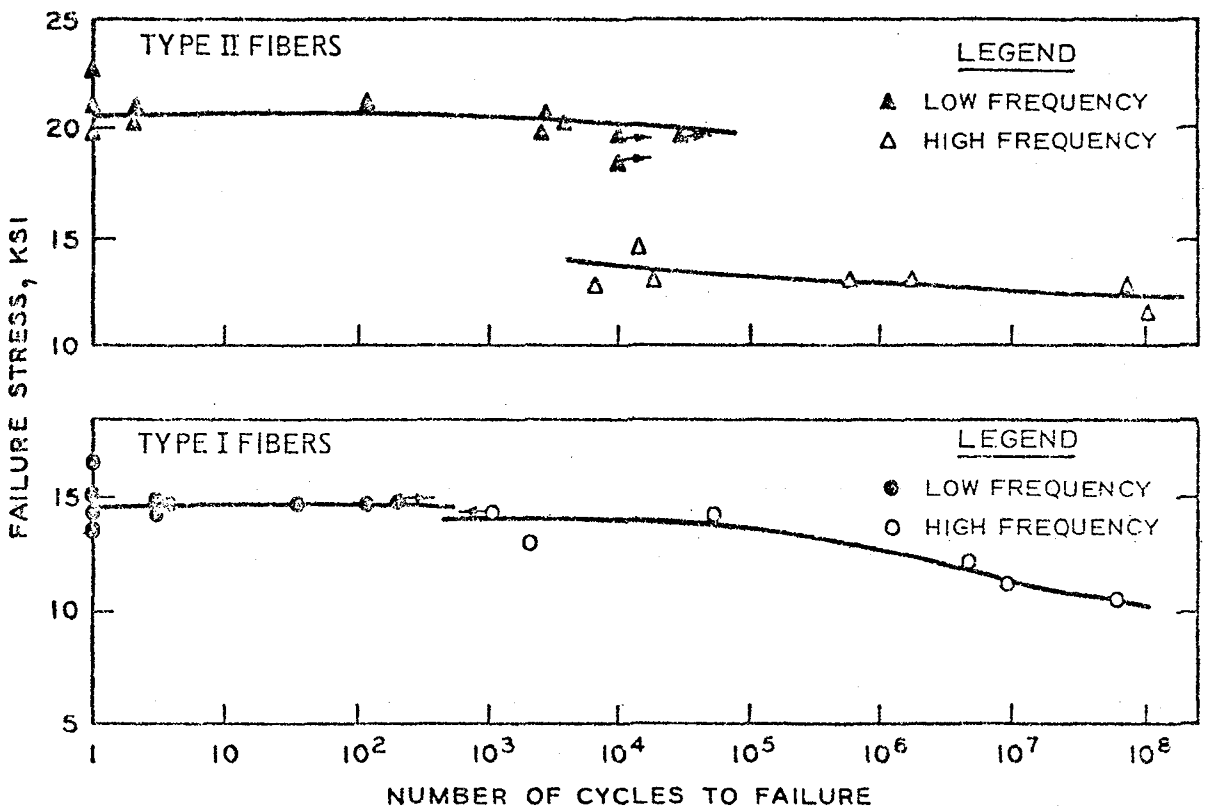


FIGURE 48 TENSILE STRESS VERSUS STRAIN RELATIONS FOR CONTINUOUS CARBON FIBER REINFORCED CEMENT COMPOSITES

(1 psi = 6.9 kN/m^2)



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 49 FATIGURE LIFE OF GRAPHITE FIBER REINFORCED CEMENT

(1 ksi = 6900 kN/m^2)

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 (713.6 and 815.5 kg/cm^2) 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.

Only small improvements (1.2 to 2.6 times the no-fiber strength) have been noted in impact strength of carbon fibrous cements (Walton and Majumdar, 1975). Combination of carbon fibers with other lower modulus fibers has shown substantial improvements.

Though carbon fibrous concrete does provide some future hope, the existing data are not enough to make a decision for large-scale field application. More research in the laboratory to evaluate the properties is needed.

4.3 POLYMERIC MATERIALS FOR CONCRETE

An enormous amount of literature currently exists and more is being written on cement-polymer materials. No attempt is made in the following pages to dwell upon the manufacturing technology of such materials. It may be remarked that no difficult unsolved technical problem remains in this area, neither in the scaling up of laboratory experimental findings to field testing nor in the appropriate manufacture for enterprise. The predominant problem is economy. The winter season of 1973-74 has brought with it abrupt changes to economic conditions everywhere in the world and continues to do so; however, no economic considerations have been included. The three established types of cement-polymer composites have been included and are the following: 1) Polymer Impregnated Concrete (PIC); 2) Polymer Portland Cement Concrete (PPCC); and 3) Polymer Concrete (PC).

4.3.1 Polymer Impregnated Concrete

Polymer imprenation of concrete means that one takes ordinary concrete, in itself a composite material of discrete aggregate particles embedded in a brittle, porous matrix of cement paste, and improves its structural characteristic strength, durability, etc., by filling the pores of hardened matrix. Concrete impregnated with a monomer such as methyl methacrylate followed by in-situ polymerization has proved to have large improvements in strength -- both in compression and tension -- in comparison with ordinary concrete. The durability and freeze-thaw resistance improves considerably. The PIC generally has an excellent resistance to chemicals. PIC generally shows a gradual reduction in strength with rise in temperature (most vinyl polymers have a glass transition temperature of about 200°F (93.3°C)). Exposure to aggressive agents such as water or brines and elevated temperatures may drastically reduce the useful temperature range, as chemical attack is greatly accelerated. Surface burning characteristic tests on PIC with ASTM designation E84-70, based on 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 the PIC materials and a smoke density factor of 0 to 15 (Carpenter, 1973).

Chen and Jorgenson (1974) comment that in its present form, the PIC is expected to find only limited application because even though the ultimate strength of PIC is high, no plastic yielding or ductility has been observed before sudden and explosive failure. If randomly oriented steel fibers are included in a PIC, the fibers are pulled out of the concrete mix when the specimen reaches a certain stress level and thus contributes little to either strength or durability (Mattisson, 1971, 1972). The brittle behavior of PIC impregnated with polymethyl methacrylate (MMA), can be improved by various manometer combinations of MMA with polybutylacrylate (BA). The higher the percentage of BA, the greater the ductility; but this ductility is achieved at the cost of decrease in strength and modulus of elasticity. Further research is needed to determine an optimum percentage combination of MMA and BA for a better co-polymer system. Table 16 and Figures 50 and 51 show the typical behavior.

Impregnation techniques (Mehta et al., 1975) have been developed to achieve penetration up to 4 in. (10 cm).

4.3.1.1 Sulfur Impregnation

The impregnation of portland cement concrete with sulfur (SIC) has been reported to yield compressive strengths as high as 2.5 X 10^4 psi (170 MPa)(a value higher than for typical PIC) with a loading of only 8 percent of sulfur by weight (9 percent by volume) (Thaulon, 1974). In addition, since sulfur is about 30 times cheaper than MMA, it is of interest as an alternate to PIC. In view of the good freeze-thaw resistance and low water permeability of sulfur concrete (Malhotra, 1973) and the excellent resistance of sulfur to acids and salts (Berman et al., 1972), sulfur may have potential applications in pavements, provided that no steel corrosion problems arise due to side reactions with water and oxygen.

Presently available methods for impregnation require the following steps: drying, cooling the deck, impregnation, and polymerization of monomer in the pores of concrete. All of the four steps are expensive and time-consuming. For example, drying at a low temperature -- say, 250°F (120°C) -- requires an inordinately long time and consequently high costs; on the other hand, drying at high temperatures -- say, 600°F (315°C) -- reduces the time drastically, but may on a large scale be limited by expansion or cracking of the structure.

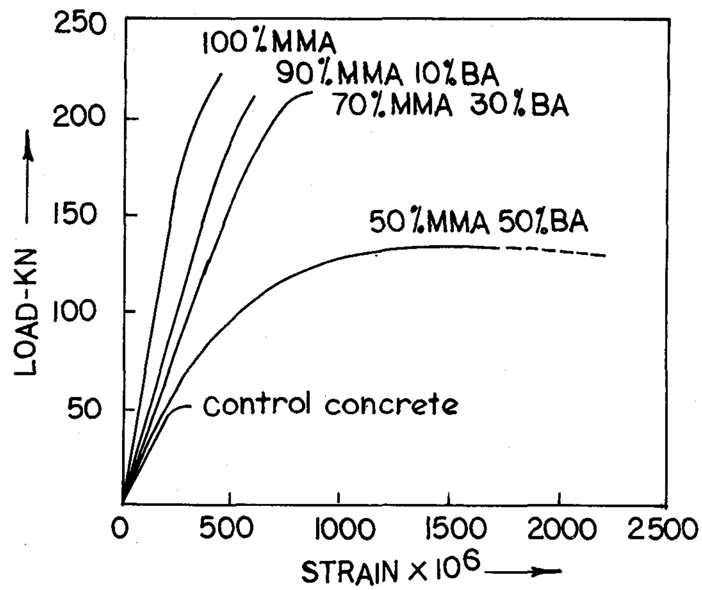


FIGURE 50 TENSILE LOAD-STRAIN CURVES

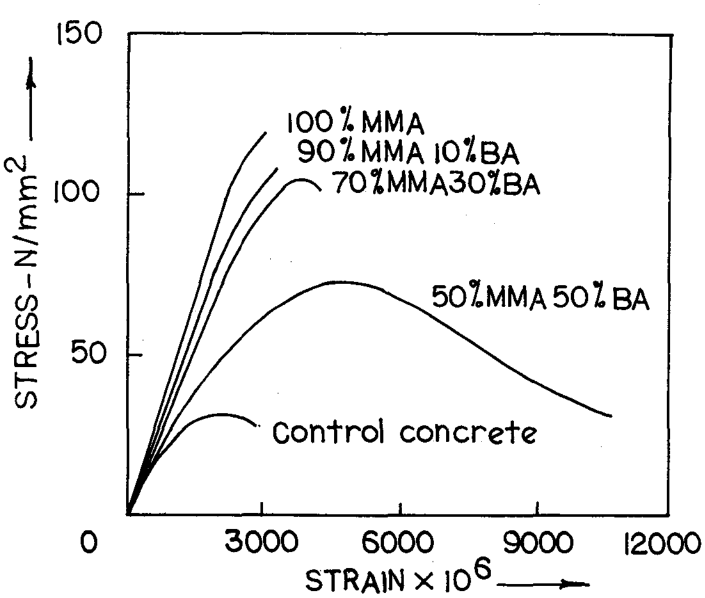


FIGURE 51 COMPRESSIVE STRESS-STRAIN CURVES

Potential advantages of using sulfur are that it melts in the range of 235°F-248°F (113°C-120°C) and the viscosity of the molten sulfur remains relatively low, from a 12.5 centipose (12.5 MPa) at 248°F (120°C) to 6.6 centipose (6.6 MPa) at 320°F (160°C). Above this temperature, the sulfur becomes highly viscous due to polymerization (probably undesirable in this case). Also in the impregnation of a pavement with a monomer such as MMA, the high vapor pressure and flammability of the monomer requires that the pavement be allowed to cool down before impregnation. Thus, an enormous amount of energy is lost -- energy that could be utilized to reduce the viscosity of materials such as sulfur (after drying of the concrete) for easy penetration into the concrete. Moreover, such energy could be conserved if drying could be effected concurrently with impregnation.

Mehta et al. (1975) proposed to combine the three steps of drying, cooling, and impregnation into one, and to eliminate the polymerization step necessary for a monomer. The basic idea is to use sulfur to simultaneously dry and impregnate concrete by covering the deck with molten sulfur. The underlying hypothesis is that sulfur will act as an effective heat transfer medium and dry the deck in advance of the penetration front. Of course, the portion to be impregnated must be kept at a temperature high enough to avoid solidification in the pores.

When a slab is heated from the surface, the moisture is seen to migrate from the hot surface down to the cool underside, and to condense in the cooler region (Dahl-Jorgensen et al., 1974). It may be supposed that as the water migrates downward from a surface heated with molten sulfur, the molten sulfur may follow the water migration due to the suction created in the pores by the moisture migration. If so, the time required for the process should be equivalent only to the drying time.

Tables 17 and 18 provide water absorption and strength characteristics of laboratory tested specimens, and Figure 52 shows the strength of sulfur impregnated specimens.

4.3.2 Polymer-Cement Concrete

Polymer cement concrete is prepared by premixing or post-mixing the concrete by polymerized materials like latexes, polymer solutions and resins and monomers.

Polyvinyl esters, polyacrylics, polyvinyl chlorides and polystyrene are the ernioplastic latexes used in mixing with concrete. Polyvinyl esters are susceptible to rapid alkaline degradation, forming acetic acid, carboxylic acids and polyvinyl alcohol. As such, the concrete made with this latex has improved properties when dry, and rapidly loses strength when wet. Polyacrylics are slightly better in terms of water solubility, but their slow alkaline degradation necessitates dry curing and limited exposure

TABLE 17

STRENGTH OF IMPREGNATED MORTAR SPECIMENS*

IMPREGNANT	SPECIMEN	% LOADING	COMPRESSIVE STRENGTH (psi X 10^3)
Sulfur	29	10.2	1.50
	30	10.1	1.48
	31	11.5	1.53
Sulfur-Tar (80-20)	39	11.6	1.14
	40	11.9	1.05
	41	10.7	1.18
Tar	43	7.3	0.64
	44	6.7	0.64
	45	6.9	0.64
Control Specimens	48	-	0.65
	49	-	0.77
	50	-	0.70

*Conversion 1 psi = 6.9 kPa.

TABLE 18

WATER ABSORPTION OF IMPREGNATED MORTAR SPECIMENS

IMPREGNANT	SPECIMEN NO.	AVERAGE[a] PERCENT LOADING	AVERAGE WATER ABSORPTION (gm)	AVERAGE[b] PERCENT REDUCTION IN WATER ABSORPTION
$Ba(OH)_2$ Solution[c]	1-7	2.3	2.02	47
Molten Sulfur[d]	8-14	12.5	0.03	99
	29-33	10.5	0.06	98
Tar[e]	15-21	6.8	0.10	97
Sulfur-Tar (80-20)	22-28	11.2	0.05	99
Water (Control)	34-38	7.8	3.84	--

[a] Percent loading = $\left[\frac{\text{impregnated weight-dry weight}}{\text{dry weight}}\right]$ X 100; immersion time = 24 hours.

[b] Percent reduction in water absorption = $\left[\frac{\text{water required for saturation-water absorbed}}{\text{water required for saturation}}\right]$ X 100.

[c] $Ba(OH)_2$ dissolved in its own water of crystallization; surface of concrete specimens severely attacked by the solution.

[d] Specimens 8-21 and 34-38 immersed in water for 24 hours; the remaining specimens immersed in water for 72 hours.

[e] Tar used was AASHO Specification M-214-65; specific gravity, 1.08.

Source: Mehta, et al.

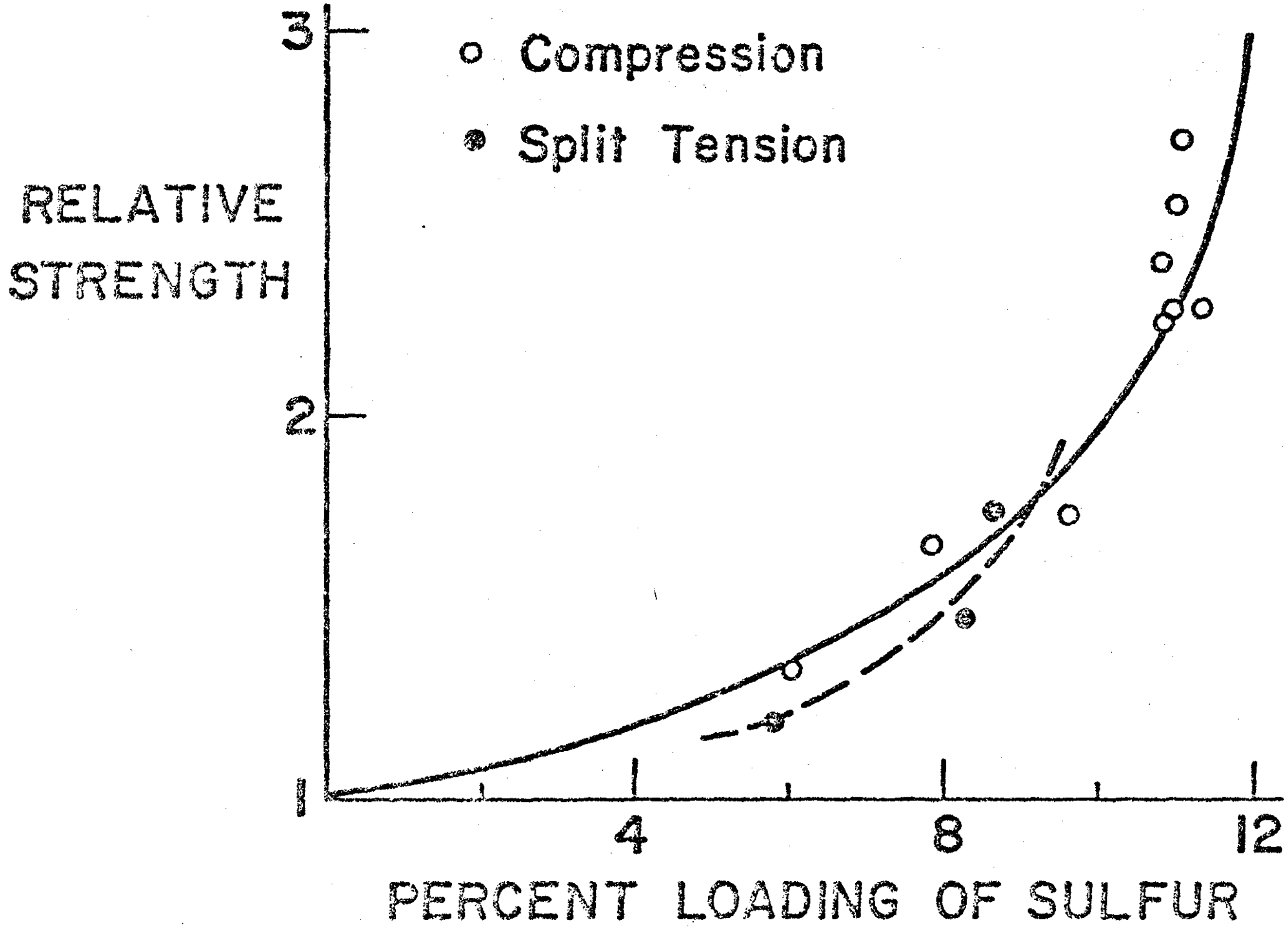


FIGURE 52 RELATIVE STRENGTH OF SULFUR IMPREGNATED SPECIMENS COMPARED TO CONTROL CONCRETE AS A FUNCTION OF SULFER LOADING BY WEIGHT

to moist conditions in practical applications. Both the above materials can improve by mixing with suitable co-polymer. A special polyvinyl chloride called Polyvinylidene chloride-PVC Copolymer (Souran) is commercially available and produces a polymer-cement concrete of superior mechanical properties and is durable under moist conditions (Dow Latexes, 1973). Similarly, a thermoplastic styrene butadiene latex formulation is commercially available and has improved durability and moisture resistance.

Elastomeric latexes are available and are more flexible but have lower strength than thermoplastic latexes. Mostly synthetic rubber is used currently.

Polymer solutions (i.e., water-soluble polymers of various kinds) have been added to fresh portland cement concrete to improve strength and water-tightness of concrete. Addition of vinyl monomers directly to the wet concrete mix followed by a later polymerization has so far not been very successful. Nutt (1970) describes a concrete called "Estercrete", which is produced by a polymer/cement ratio of 0.3 (or higher) together with a water-soluble redox catalyst for condensation. When mixed with water, a catalyst is activated, polymerization occurs and the cement hydrates. Celanese Coatings Company (1972) has developed an epoxy portland cement concrete; such developments do exist in Japan and many other countries.

The presence of a polymer phase provides desirable properties in concrete: its ductility and durability is increased along with the superior adhesive properties. However, several mix designs and many different latex materials change the property factors such as dispersed particles. Type and amount of emulsifying agents (to increase the stability of the latex), its coalescence on drying, etc., will affect the properties (Solamator, 1967).

The tensile and flexural strength due to increase of ductility is increased along with increased strain at failure (Figure 53). Isenburg and Vanderhoff (1974) believe that a major benefit of the polymer in relation to strength is due to a strong cement aggregate bond. The polymer itself halts propagating microcracks and holds existing microcracks together. Table 19 presents some properties.

As a result of lower water-amount ratios and partial filling of pores by a polymer (also sealing of existing pores by polymer film), the porosity is reduced (Chen et al., 1973); so permeability, water absorption and water vapor transmission are reduced, thereby making the product more durable.

Resistance to chemicals depends on the polymer and chemical, but suitable polymer can be selected to fight any environmental or locational chemical effects. Excellent freezing and thawing resistance has been observed of polymer-cement concrete.

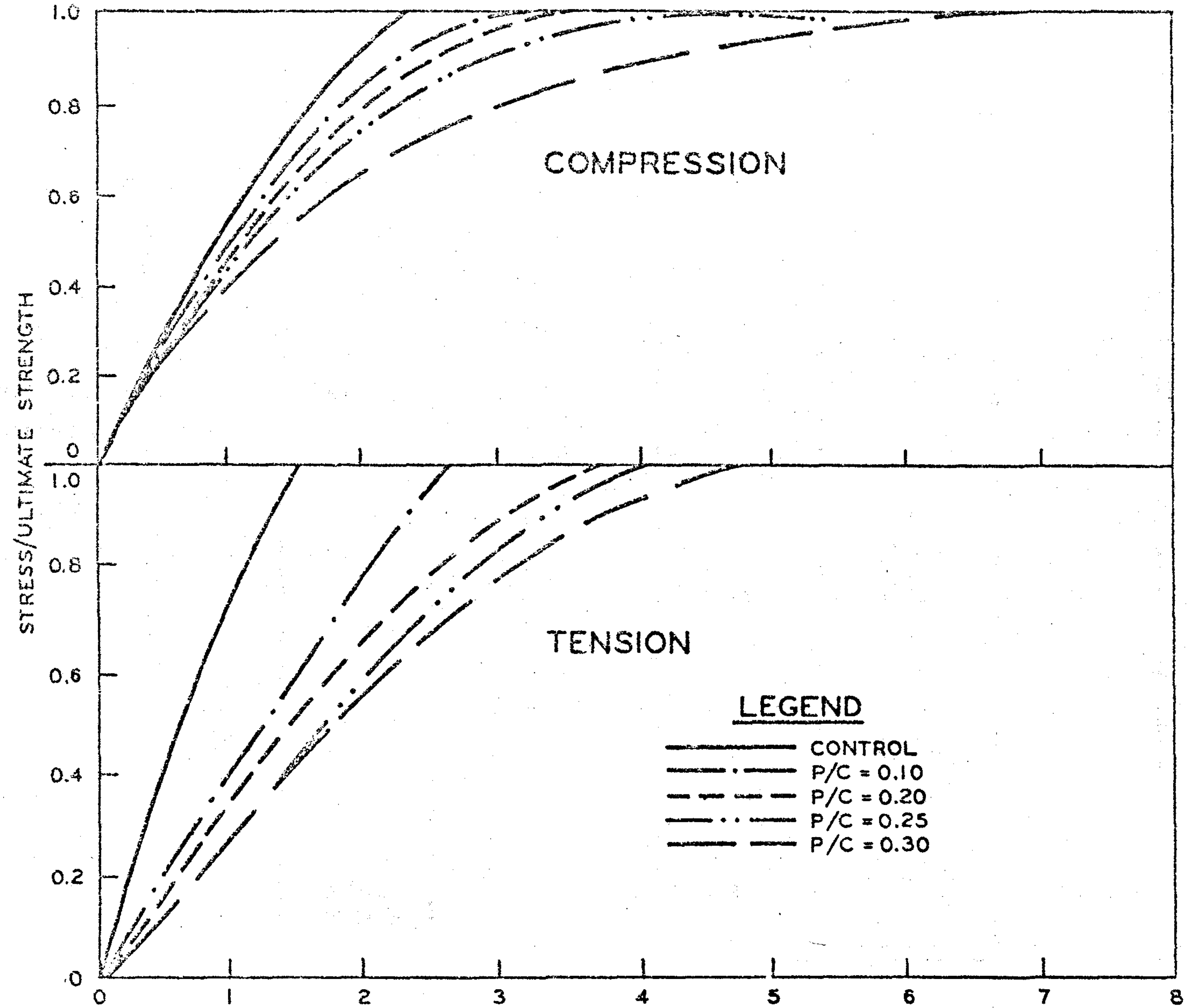


FIGURE 53 STRESS-STRAIN CURVES FOR POLYMER MODIFIED CONCRETE

TABLE 19

TYPICAL MECHANICAL PROPERTIES OF PIC

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*	3.0-5.0	10,240	675	--	4.2	--
Vinylidene chloride*	1.5-2.8	6,650	370	--	3.0	--
t-butyl styrene*	5.3-6.0	18,150	1,445	--	6.4	--
60% styrene + 40% MPTMA*	5.9-7.3	17,140	910	--	6.3	--

Note: Concrete dried at 105°C overnight and was radiation polymerized.

*Dried at 150°C overnight.

Source: Kukacka, et al., 1973.

Note: 1 psi = 6.9 kN/m^2

Abrasion and impact resistance as well as toughness is significantly increased (Table 20). The polymer-cement concrete shows excellent adhesive properties -- a valuable property for use in overlays.

While information on dynamic and fatigue strength does not exist in literature, intuitively the material should have good dynamic and fatigue resistance. However, there are three disadvantages: 1) some latex modified concrete loses strength when immersed in water or exposed to high humidities (Tables 21 and 22); 2) modified concretes, containing thermoplastic polymers, will lose mechanical properties at higher temperatures; and 3) increased creep behavior is observed (at 50°C); specimens of vinyl chloride-vinylidene co-polymers failed under a stress equal to 0.33 of the ultimate static strength.

The problem of losing strength under high humidities has now been overcome by use of co-polymer formulations and the availability of various commercial mixes. It is believed that for a set of environmental conditions, a good polymer modified concrete can be designed with the current state-of-the-art.

4.3.3 Polymer-Concrete

In polymer-concrete, only a synthetic resin such as a polyester or epoxy constitutes the binder. The aggregate used in synthetic concrete is similar to that of portland cement concrete. Polymer-concrete exhibits distinct advantages as follows (Knab, 1969):

1. Very high tensile, compressive and flexural strengths, two to three times that of ordinary cement concrete;
2. Rapid rate of cure -- many synthetic systems achieve near full strength in several days or less;
3. Excellent impact and abrasion resistance;
4. Very high resistance to freeze-thaw and general weathering action;
5. Excellent chemical, solvent water and salt-spray resistance;
6. High-strength bonding characteristics with most highway materials.

The disadvantages are:

1. Some polymer-concrete systems are limited to dry conditions;

TABLE 20

MECHANICAL PROPERTIES OF LATEX MODIFIED MORTARS

	CONTROL	STYRENE-BUTADIENE	SARAN[a]	ACRYLIC	PVAc
Compressive strength (psi)	4500 (5800[b])	4800	8430	5700	3700
Tensile strength (psi)	310 (535[b])	620	910	835	700
	380	830			
Flexural strength (psi)	610 (1070[b])	1430	1820	1835	1840
Modulus of elasticity (10^6 psi)	3.40	1.56	2.25	--	--
Shear bond strength (psi)	50-200	650[c]	>650	>650	>650

Note: All mixtures had a sand/cement = 3, polymer/cement = 0.20, and were dry-cured 28 days at 50% R.H.

[a]Vinyl chloride-vinylidene chloride co-polymer.

[b]Moist-cured 28 days.

[c]Exceeds shear strength of mortar.

Sources: Dow Chemical Co., Tech. Bulletin, 1975.
Rohm & Hass Co., Bulletin, 1970.

Note: 1 psi = 6.9 kN/m^2

TABLE 21

EFFECT OF IMMERSION IN WATER FOR 7 DAYS ON THE STRENGTH OF LATEX MODIFIED MORTARS

	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
Styrene-butadiene	4950	4100	600	350	1425	925	>650[b]	350
	4800	3680	--	--	1730	770		
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.

[b]Exceeds shear strength of mortar.

Source: Kukacka, et al., 1973.

Note: 1 psi = 6.9 kN/m^2

TABLE 22

WEAR RESISTANCE OF LATEX MODIFIED MORTARS

	CONTROL	STYRENE BUTADIENE	ACRYLIC	PVAc
Impact strength (in. lb.)	6 (7)*	19	22	16
Abrasion resistance (% weight loss)	24 (5)*	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.

*Controls were moist-cured 28 days.

Source: Rohm & Hass Company, Technical Bulletin, 1970.

2. Creep increases with temperature. Increases in creep of several hundred percent have been reported within range of anticipated summer temperature fluctuations -- 18°C to 49°C (Hardy Shell, 1972);

3. Loss of structural integrity at high temperatures.

4.3.4 Conclusions

Polymer-portland cement concrete is easiest to produce and to place compared to polymer impregnated concrete and polymer-concrete. We feel that from the point of view of availability of polymers, the use of PPCC and/or PIC as a top sealing and wearing course in a pavement system is advisable. The sulfur-impregnated concrete appears more promising. Except for patch repairs or restricted areas, the use of polymer-concrete on a mass scale may not be desirable.

It would be desirable to optimize the PPCC and/or sulfur-impregnated concrete by mixing with fibrous steel concrete in such a way that the top 2 in. (25.4 mm) of the total fibrous steel concrete slab may have either PPCC or PIC. This would provide the benefit of ductility, water-tightness, sealing of existing cracks as well as to arrest the propagation of new cracks.

4.4 WAFFLE, CORRUGATED AND ANCHORED PAVEMENTS

4.4.1 General

The ideas presented in this section were developed to remedy some specific problem areas that arise when rigid pavements consisting of portland cement concretes are constructed. For instance, as concrete hardens, it tends to shrink and cause tensile stresses to form, initiating and propagating cracks. Other detrimental effects (intrusion of water and foreign material, faulting, pumping, etc.) are then likely to occur.

Another problem that must be considered is pavement expansion and contraction due to changes in temperature. Traditionally pavements have been designed with special joints, especially at structures (bridges), that allow changes in length in the longitudinal direction. The idea is to control rather than prevent these effects.

Some of the concepts that have been devised include waffle-patterned, ribbed, or corrugated subsurfaces; anchored pavements, and pavements with variable cross sections and reinforcement patterns.

4.4.2 Waffle Patterns

Very limited information exists in the literature concerning waffle or corrugated subsurfaces or highway and/or airport pavements. That which is available comes primarily from Europe.

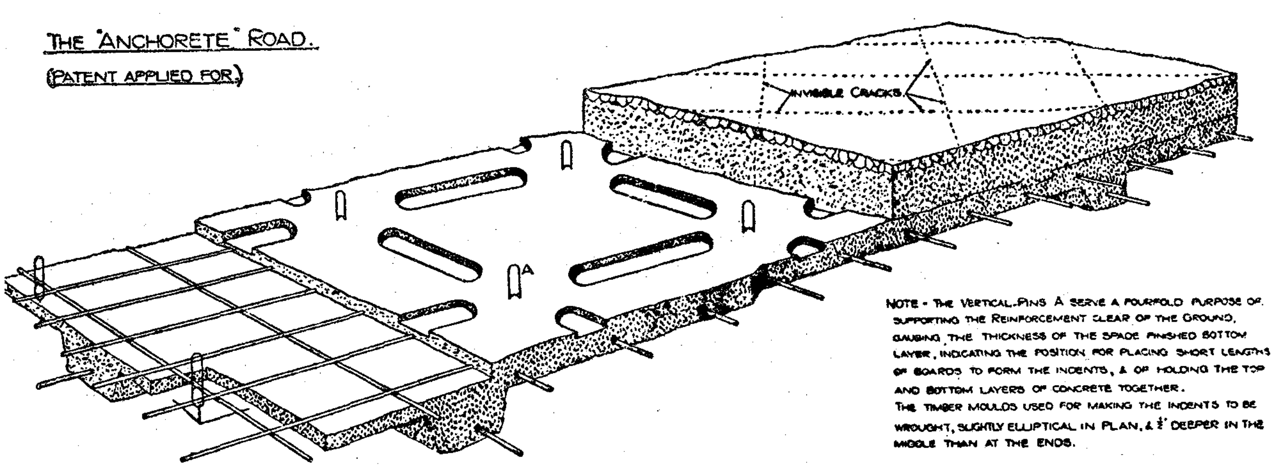
Perhaps the first mention of waffle patterns comes from England. In 1931, C.P. Courtenay and J.H. Walker (Walker patented the idea) developed the system and the underlying theory. Walker called the system "Anchorete" (Road and Road Construction, 1931); several pavements were constructed in Great Britain in the 1930's that apparently were crackless over a period of at least 5 years. (It should be kept in mind that traffic loads were not what they are today.) However, the "Anchorete" system can be modified to meet present and future demands.

Courtenay used unreinforced concrete that was "anchored" to a hard pan gravel foundation, the details of which are unknown at the present time. This pavement inhibited expansion and construction that inevitably results in crack formation.

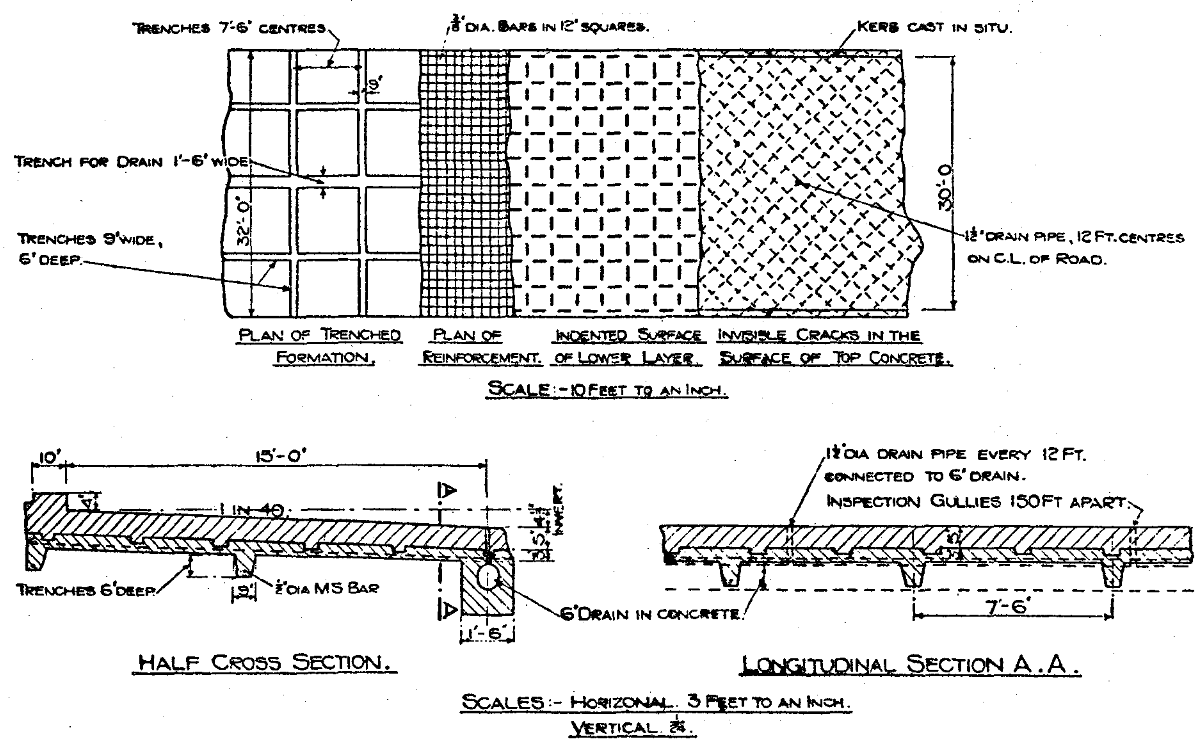
"Anchorete" confines expansion and contraction effects to small areas and produces small invisible cracks that do not damage the stability of the concrete. The system involved gridding the subgrade with 4- to 6-inch-deep (102 to 152 mm) trenches that are on 7-foot (2.1 m) centers. Reinforcement is placed (0.59 percent both longitudinal and transverse) and concrete is poured, creating a continuous base material. Keys are formed in this base to hold the surface course (see Figure 54). An unreinforced wearing course is then placed, keying into the base.

As the surface course hardens, it contracts. Since it is keyed to the base, however, contraction is inhibited, resulting in the numerous small cracks mentioned. The base is somewhat insulated from large thermal gradients and as a result does not expand and contract as much, thereby reducing the total number of cracks considerably. The upper layer can then expand and contract, the cracks provided from curing acting as many minute expansion and contraction joints (Walker, 1931). With present technology, it is possible to seal these cracks with some sort of elastic polymer that can impregnate the upper portion of the wearing course, thus not only preventing water to penetrate the slab, but also providing an elastic crack filler during a contraction cycle.

Waffle or ribbed pavements have been used elsewhere in Europe. Many experimental pavements of prefabricated waffle-patterned slabs, either prestressed or unstressed, have been constructed in the USSR. Since these slabs are prefabricated, they would not key with the base as well as poured-in-place pavements. However, they seem to work well on soil stabilized base courses (Mogilevich et al., 1976) either for city streets or on major highways. The prefabricated



A) CONSTITUENTS OF THE PAVEMENT



B) ROAD CONSTRUCTED AT THE SURREY DOCKS IN ENGLAND

FIGURE 54 "ANCHORETE" SYSTEM AS DEVELOPED BY WALKER

(1 in = 25.4 mm; 1 ft = 0.3048 m)

construction calls for some sort of leveling course (½ to 1-inch (12.7 to 25.4 mm) thick) to be placed before the slab is vibrated into place. The leveling course serves the dual purpose of leveling the surface for the slabs and helping to bond the ribs to the base (this requires an asphalt/sand mix or sand/cement mix).

Other types of construction include grid foundations on which prefabricated slabs are placed. Pavements of this type have been constructed in Switzerland and the USSR. However, a knowledge of their performance and construction details could not be found.

Unfortunately, only qualitative data on these types of pavements exist. Evaluation is almost impossible without further analysis of field data collected from experimental pavements constructed in a variety of soil conditions and climatic conditions.

4.4.3 Anchored Pavements

Anchored pavement sections have been used in Texas on jointed concrete pavements (JCP) and continuously reinforced concrete pavements (CRCP). They are used near structures to absorb the effects of longitudinal expansion or contraction of pavements (Shelby and Ledbetter, 1962; McCullough, 1971).

The terminal anchorage system utilizes the passive bearing and shear resistance of subsoils to transfer compressive stresses in the pavement that arise from expansion or tensile stresses due to contraction. Lugs transverse to the pavement extend beneath the pavement to transfer the stresses (Figure 55). It was assumed that only 500 ft (152 m) of CRCP contributes to the expansion of the terminal section at a structure (bridge). The remainder of the pavement is restrained by friction with the subbase. Based on this assumption and collected data, a relation was developed linking movement at an expansion joint with change in air temperature as follows:

$$\Delta X = \frac{0.1253 \left(\frac{L}{|G| + 1} \right)^{0.107}}{C^{2.027} \, (N + 1)^{0.312}} \, \Delta T$$

where

ΔX = the total movement for a given temperature

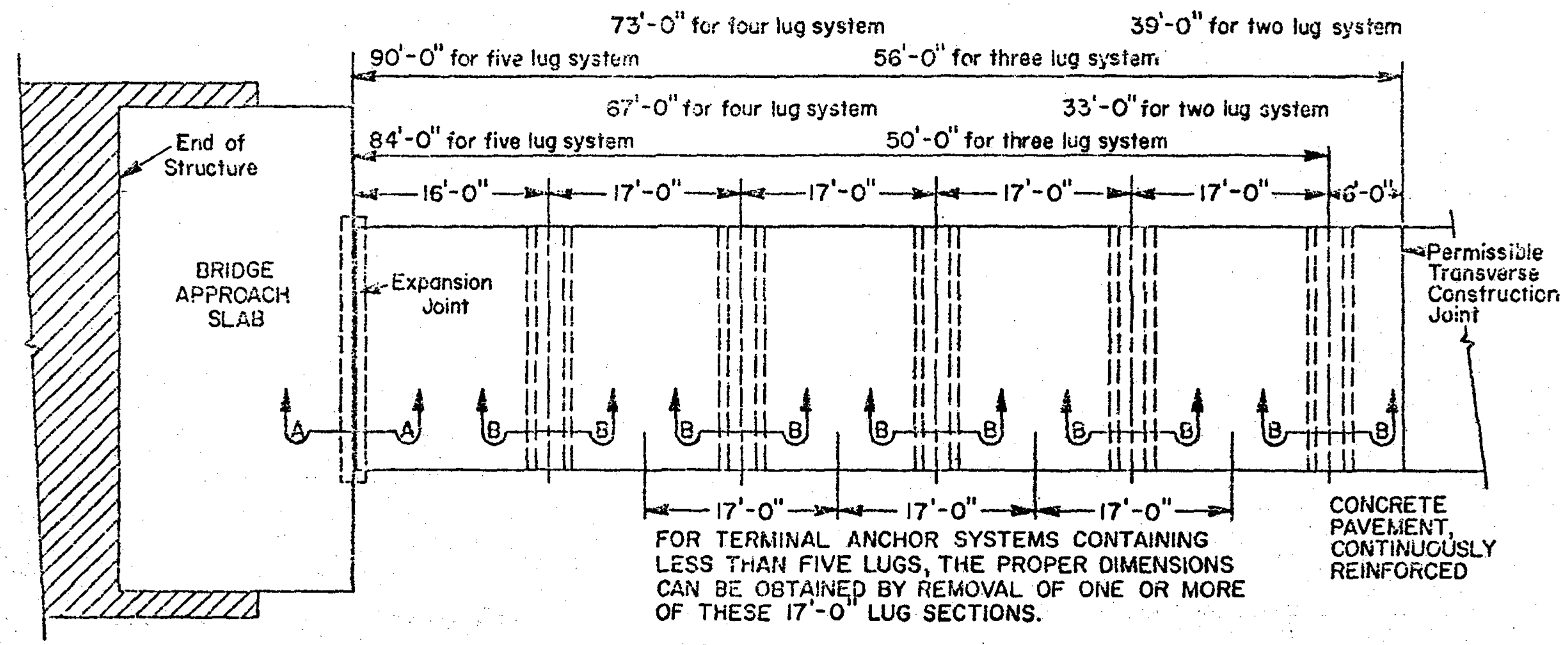
ΔT = change in air temperature in ^{o}F

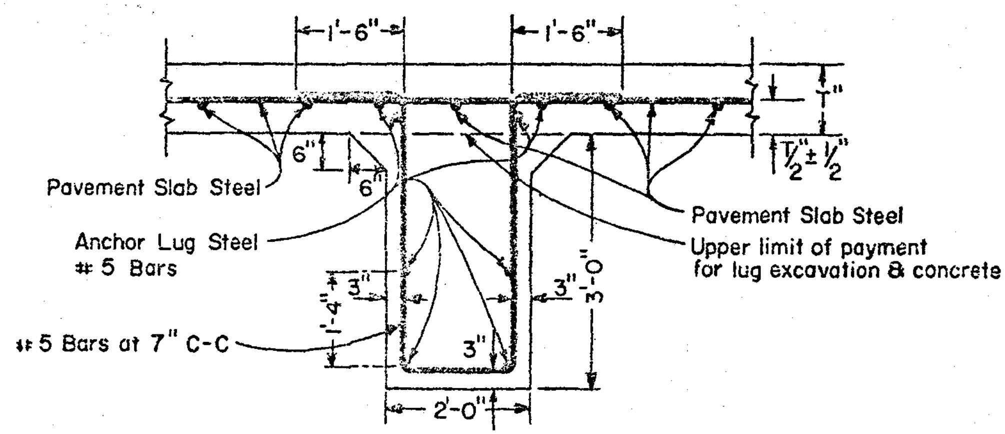
C = subbase coefficient of friction

N = number of lugs

L = length of slab contributing to end movement, ft

$|G|$ = absolute value of percent grade





ANCHOR LUG DETAIL

FIGURE 55 TYPICAL LUG DESIGN - CRCP

(1 in = 25.4 mm; 1 ft = 0.3048 m)

The constants were developed from a multiple regression analysis with coefficients of friction of the subbase ranging from 1.35 to 2.65. Climatic effects were eliminated through the analysis. Pavement age had little effect on elongation (Figure 56). Figure 57 provides a plot of rate of end movement for a different number of lugs and various subbase materials.

This idea can be extended for use in pavements (especially CRCP) between structures as well as at structure locations. The controlled movement requires conventional expansion joints that operate in a range of expansions much lower than normal. Instead of a joint every ¼ mile (400 m), expansion joints can be installed every 2 to 3 miles (3 to 5 km), depending on the lugs used (number, size, etc.).

4.4.4 Additional Thickness or Reinforcement

A pavement of variable cross section thickness or of variable amounts of reinforcement can serve several purposes. Firstly, stiffening a pavement through the use of a higher reinforcement ratio under wheel locations can spread the load over a large area, thus reducing stresses in the base and subgrade. Secondly, increasing the slab thickness at the edges and transverse to the edges stiffens the slab and can reduce curling (this is also a possible feature of waffle slabs). Finally, more complex undersurface contours can promote better contact with the subbase. Slab curling can cause this loss of contact. Since stability of a pavement is directly related to the properties of underlying layers and contact with these layers, methods to insure good base contact of a rigid concrete slab should be studied. Mogilevich (1976) presents some solutions to this problem from the standpoint of prefabricated pavements that can also be applied to cast-in-place pavements: low amplitude, high frequency vibration, pressure grouting, slab contours (see Figure 62), etc.

A dual reinforced pavement, that is two layers of reinforcement, has been used in experimental test sections of continuous reinforced pavement (Wagner, 1977). The pavement contains two layers of welded wire mesh and has performed well for over 30 years. The two layers can also reduce the effects of stress reversals or other adverse stress conditions.

4.4.5 Conclusions

Consideration and evaluation of some of the concepts reported are warranted. Especially promising types of pavements include waffle or grid systems and increased thickness of pavement edges to reduce curling effects.

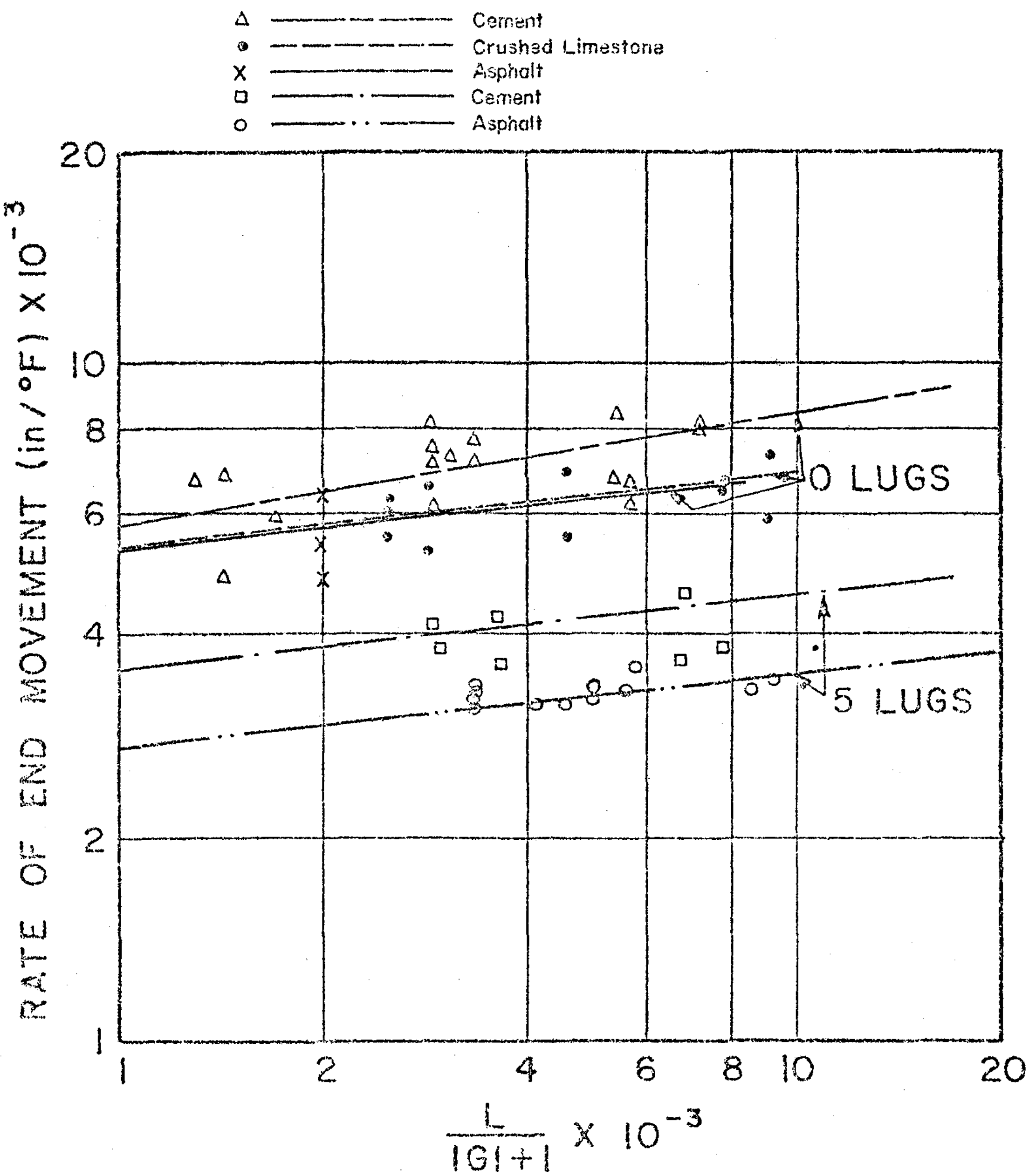


FIGURE 56 EFFECT OF PAVEMENT AGE ON END MOVEMENT

(1 in = 25.4 mm; $^{o}C = (^{o}F-32)/1.8$)

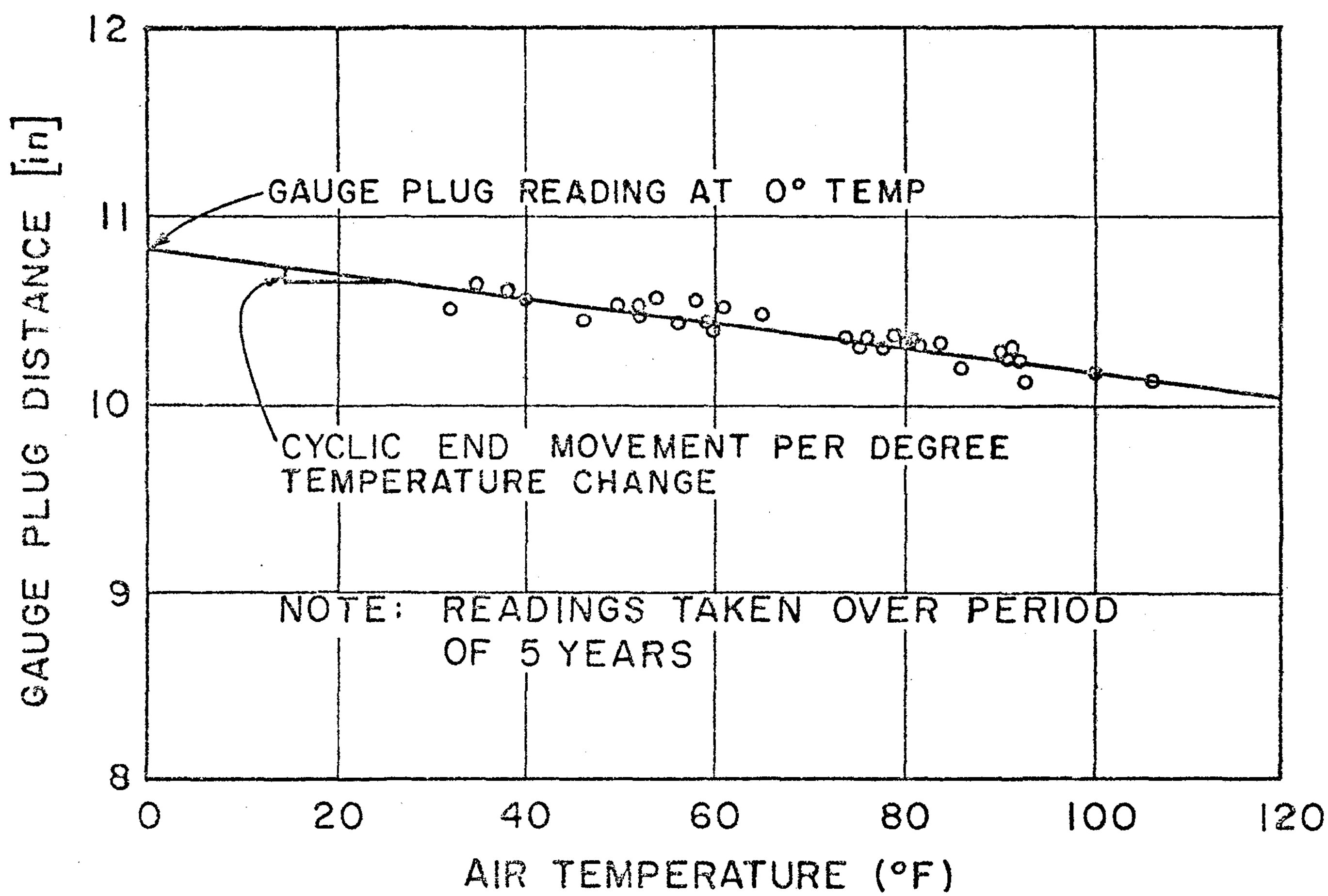


FIGURE 57 CYCLIC END MOVEMENT VS. PAVEMENT LENGTH - PERCENT GRADE TERM FOR ASPHALT-STABILIZED, CEMENT-STABILIZED, AND CRUSHED LIMESTONE SUBBASES

(1 in = 25.4 mm; °C = (°F-32)/1.8)

4.5 PAVEMENT ON PILES

The heavy traffic prevailing near the metropolitan areas provided a hint to utilize some of the ideas suggested for airfield pavements. The pavements on piles were suggested for strengthening in-situ pavement systems by installing piles, or pile-like inclusions in weak subgrades (e.g., at LaGuardia Airport). Most of the work done in this area has been analytical, and in very few places where the pavement on piles has been constructed, those analytical techniques have hardly been used. In practice, a "crushed rock pile cap" on top of wooden piles has been specifically used for the support of storage tanks, some in New Jersey and probably in other places as well. It is conceivable that driving piles at some moderately spaced intervals and then constructing a compacted earth or earth and rock fill that gains part of its support from the piles without any structural or concrete slab on top of the piles could have some applications in some special situations for highway and/or bridge approach construction.

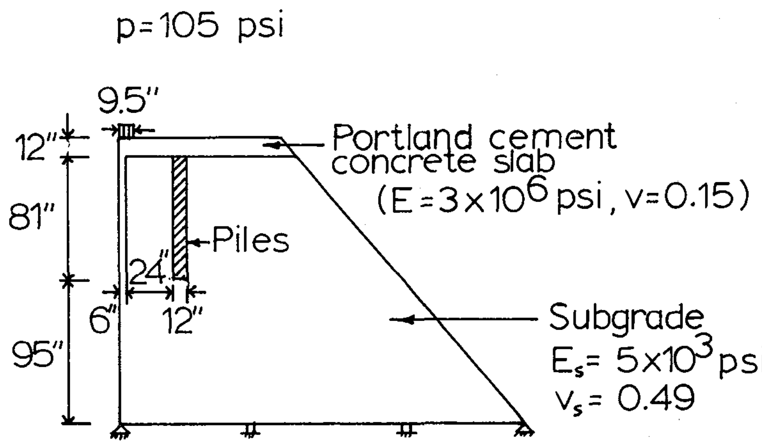
Pichumani et al. (1974) developed a three-dimensional finite element technique to analyze a pile-supported airfield pavement system in a most general form. A "Structural Analysis Program" (SAP), developed by Wilson (1970), was used to evaluate the effectiveness of strengthening the weak subgrades. Rigid as well as flexible pavements were analyzed. Figure 58 shows the geometry and material properties associated with each model. The three-dimensional finite element analysis showed that:

1. The surface deflections of both types of pavements decrease with increasing pile stiffness;

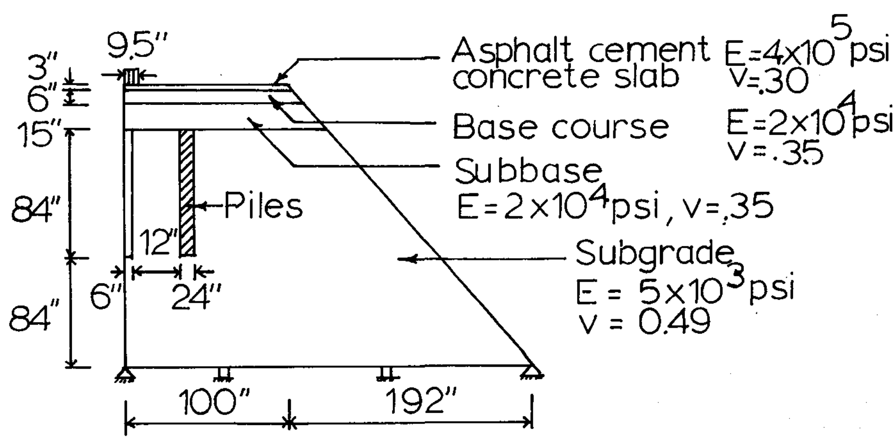
2. Subgrade stresses are reduced in regions surrounding the pile while they are increased below the pile tips. The stress reduction near the top of the subgrade is obviously helpful, while the increase in the subgrade stress below the piles may not be critical;

3. The horizontal tensile stress in portland cement concrete pavement slabs is reduced by pile inclusions.

Poulos et al. (1975) argues that the quoted cost of computer time for each case studied by Pichumani et al. is too high to be justifiable unless no other method is available. They suggested an alternate analytical technique based on equations given by Mindlin (1936), and an integration technique suggested by Poulos and Davis (1968). This approach has been extended to produce a computer program for analysis of strip footings that are supported both by piles and an elastic half space. This has also been used for analysis of pile strips on finite layers of soil by means of an approximation described by Steinbrenner (1934). The technique of analysis currently is only suitable for strips with a length/breadth ratio greater than three and for homogeneous, isotropic soil masses.



A RIGID PAVEMENT



B FLEXIBLE PAVEMENT

FIGURE 58 PILE-REINFORCED PAVEMENT MODELS
(1 in = 25.4 mm; 1 psi = 6.9 kN/m^2)

Analysis of the results of Pichumani et al. shows that under a single-wheel load of 30,000 pounds (13,600 kg), the maximum deflection was 0.018 in. (0.46 mm) without piles (soil modulus = 5000 psi (70,300 kg/cm^2)) and reduced to 0.015 (0.38 mm) with piles having a modulus of 1 x 10^6 psi (70,300 kg/cm^2) (Figures 59 and 60). Comparison also shows that the pavement with a subgrade having a modulus of 5000 psi (351 kg/cm^2) and reinforced by piles with a modulus of 1 x 10^6 psi (70,300 kg/cm^2) is equivalent in strength to a pavement with a subgrade having E_s = 10,000 psi (702 kg/cm^2) and no piles. The surface deflection reduces from 0.015 in. (0.38 mm) (soil modulus 10,000 psi (702 kg/cm^2), no piles) to only 0.011 (0.28 mm) by doubling the soil modulus (no piles). The effectiveness of piles in reducing surface deflection is clearly demonstrated.

However, the operating vehicles' motion will produce (dynamic) horizontal loads on the piles. When the pile vibrates, its stiffness is modified and damping is generated through interaction of the pile with surrounding soil. These phenomena are very complex and little understood. In earthquake work, horizontal response of the piles was studied by Penzien (1970) and few others. However, experiments have shown that the vertical response can be affected much more by the motion of the tip than is horizontal response (Novak, 1976) because the axial stiffness of the pile is very high. Inclusion of the motion of a pile tip in a rigorous analytical solution is extremely difficult. An approximate analytical approach has been used by Novak (1977) using a floating pile, which seems more applicable for the case of a pavement on piles.

To design a pile-supported pavement structure, the structure and damping constants of the soil-pile system at the level of pile head are required. The stiffness and damping of piles vary with frequency. After they have been determined, the design procedure is similar to that used for shallow foundations. Experimental and analytical work to date finds that:

1. The motion of the pile tip cannot be neglected unless the pile is extremely long or the tip rests on a very rigid layer;

2. Lack of fixity of tip reduces the stiffness and significantly increases the damping;

3. With increasing length, the stiffness of a floating pile increases, while stiffness of an end bearing pile decreases;

4. The damping of floating piles is larger than that of end bearing piles;

5. The vertical dynamic response of a footing supported by piles can be much smaller with floating piles than with end bearing piles;

6. The relaxation of pile tip improves the agreement between the theory and experiments.

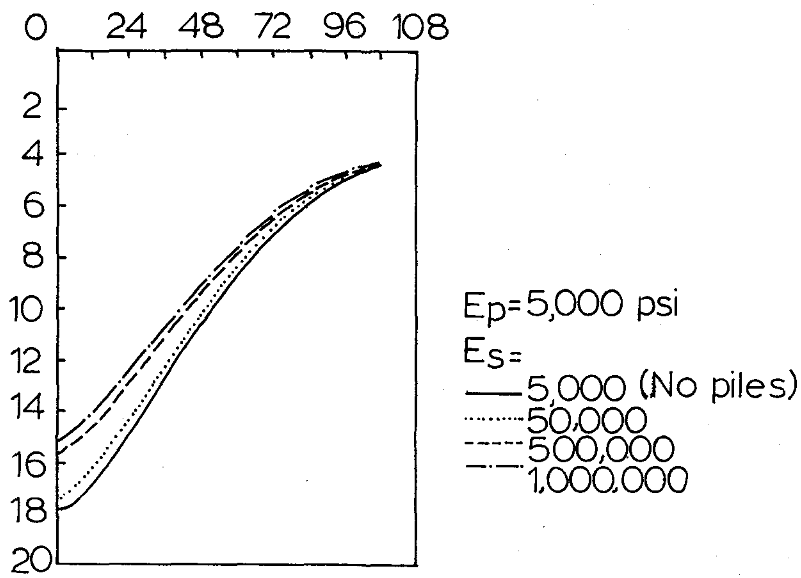


FIGURE 59 EFFECT OF PILE STIFFNESS ON VERTICAL SURFACE DEFLECTION OF RIGID PAVEMENT SYSTEM

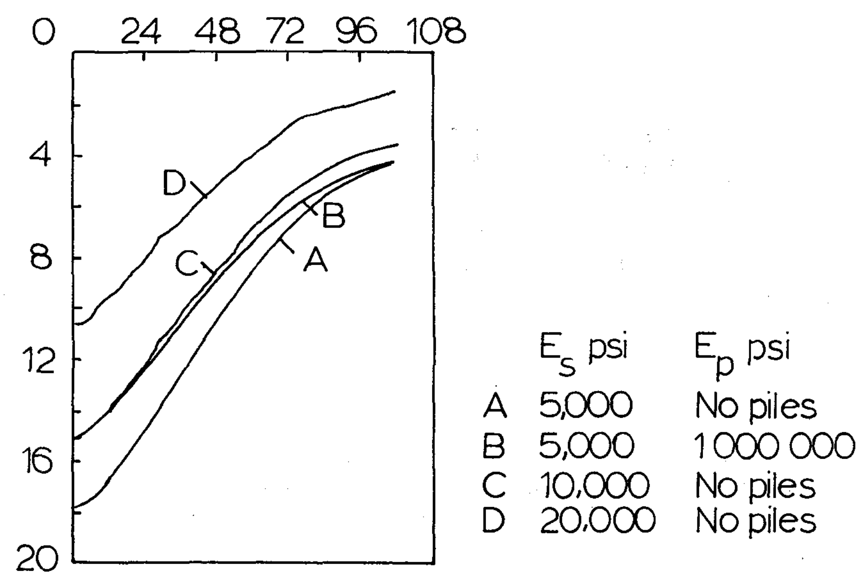


FIGURE 60 COMPARISON OF SURFACE DEFLECTIONS OF VARIOUS RIGID PAVEMENT SYSTEMS

(1 psi = 6.9 kN/m^2)

4.5.1 Conclusions

The system of pavement on piles cannot be excluded from consideration in a zero-maintenance system, especially when one considers the meagre reduction in surface deflection by doubling the soil modulus values (Figure 59). However, the system needs more study. In the next Chapter, a pavement system with piles is proposed and recommendations are made for an in-depth study.

4.6 PREFABRICATED PAVEMENTS

4.6.1 General

Prefabrication in the building industry has been proved worth-while by significantly decreasing costs and construction time, allowing greater quality control and increasing production. The actual transition from monolithic pavement construction to the installation of prefabricated pavement elements makes it possible to change the organization of the construction process as well as increase the effective use of labor forces and material resources.

Prefabricated pavement is not a new idea. Mogilevich et al. (1976) presents numerous historic examples of prefabricated pavements used throughout the USSR and Europe. In the United States, many brick and cobblestone streets are still in use, some dating back well over 100 years. These types of construction, however, were created when materials and especially labor were relatively inexpensive. Modern times call for speedy, economical construction to help reduce overall cost. Thus the size of prefabricated pavement slabs must increase to reflect this trend.

Slab materials include portland cement concrete, polymer concretes, fiber reinforced concretes, silicate concretes, asphaltic/bituminous concretes, slag/fly ash concretes, etc. The type used reflects material availability, load carrying capacity, construction techniques, cost, etc., just as it does with conventional pavements. The most promising slabs are rigid types composed of either plain, reinforced, or prestressed concretes. Due to large installation stresses, flexible materials such as asphaltic concretes are not recommended except for base courses (Mogilevich et al., 1976).

4.6.2 Type of Slabs

4.6.2.1 Reinforced Slabs

Reinforcing of structural concretes accomplishes several goals: 1) reduces pavement thickness; 2) increases slab size, thereby reducing the number of joints necessary; and 3) decreases materials required, etc. Conventional welded wire mesh and deformed bar reinforcing or newer more exotic methods of reinforcing structural concretes such as steel or synthetic fibers, polymer impregnation, etc., are available for use in reinforced prefabricated pavement slabs. The criteria for material selection are similar to those of conventional pavements.

Slab shapes and paving configurations are quite varied, ranging from simple rectangular slabs laid side-by-side to more complicated shapes that interlock (see Figure 61). However, in the interest of promoting more uniform and simple modular construction, thereby reducing costs associated with manufacture and construction of slabs, it is recommended that rectangular or hexagonal slabs be used (Mogilevich, 1976). Hexagonal slabs behave the same way in all directions due to radial symmetry and an absence of more acute corner angles. They are also less susceptible to tension in upper fibers of the slab than are rectangular slabs. However, hexagonal slabs possess some inherent flaws. They contain more joints per unit surface area than a comparable pavement composed of rectangular slabs. Construction is more involved due to special half slabs that are required along curb or shoulder lines. As a result of these problems, the use of hexagonal slabs has been left for ornate walkways or squares; rectangular slabs have been singled out for general pavement applications.

Slab cross sections are just as varied as slab shapes. Figure 62 shows some typical undersurface details including flat or concaved, edge anchored, or waffle-patterned surfaces. The variety exists to provide a choice for designers who use or encounter various different base structures or subgrade conditions. Uniform slab to base contact is essential to efficiently transfer traffic loads from the pavement surface to the subgrade.

The size of the slab is a function of many parameters as mentioned earlier. Slab sizes range from 3 to 4 ft (90 to 120 cm) square to over 15 ft (460 cm) square. The size of the slab should necessarily be directly linked to traffic lane widths to follow the modular concept of prefabrication. If a pavement width is two lanes, each 12 ft (3.7 m) wide, slabs should be 6, 12 or 24 ft (1.8, 3.7, or 7.4 m) wide. The latter case requires either very large slabs or narrow slabs laid with the long edge transverse to traffic, similar to the experimental pavement constructed in South Dakota (Hargett, 1968; Larson et al., 1972). To promote water runoff, these large slabs require a crown in the center or elevation of one side of the slab above the other.

Unstressed reinforced slabs of dimensions of 24 ft (7.4 m) tend to be relatively thick (to accept installation stresses) and hence very heavy, posing transportation and construction difficulties. It follows that slab dimensions should be decreased to a more manageable size and still insure modularity.

Very little information exists on material properties of prefabricated reinforced slabs. Mogilevich et al. (1976) gives some general properties of various concretes and steels (i.e., Young's modulus, Poisson's ratio, flexural strength, etc.), but only qualitative information on performance of pavement constructed

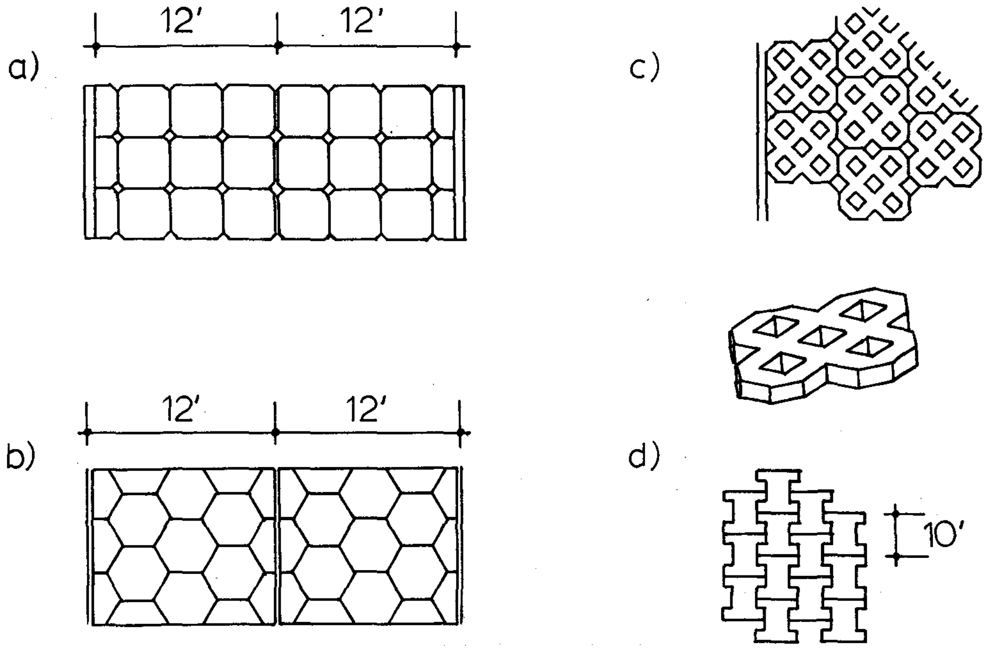


FIGURE 61 VARIETY OF SLAB SHAPES

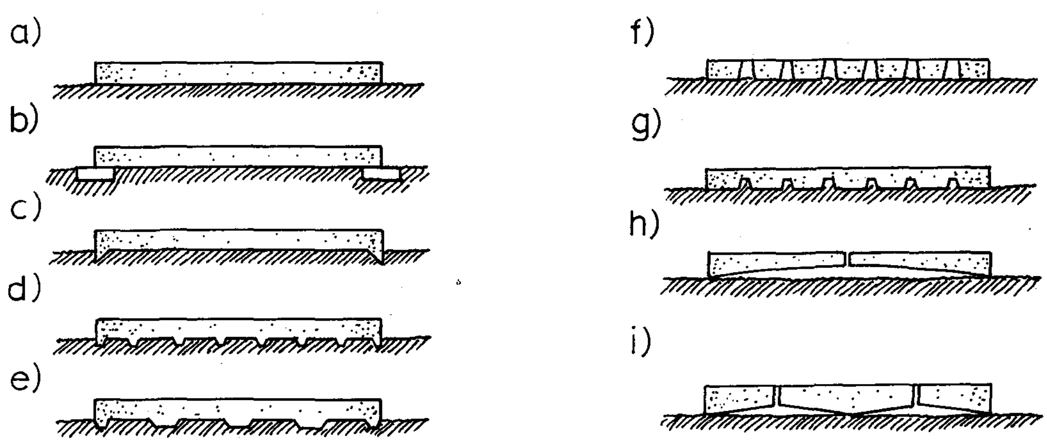


FIGURE 62 TYPICAL SLAB CROSS-SECTIONS

of various types of prefabricated elements is presented. It is believed that strength and other properties of the various prefabricated slabs of reinforced concretes are comparable with those of monolithic pavements. Sharp (1977) concludes that few problems exist from the standpoint of the structural integrity of concretes themselves; the subbase and subgrade provide the major source of weakness when failure develops in a pavement. Hence, most of the effort involved in designing zero-maintenance pavements must center around providing adequate foundations for pavements, including prefabricated pavements. The superstructure can be adequate only if the supporting structure is sound.

4.6.2.2 Prestressed/Post-Tensioned Slabs

More information exists concerning prefabricated prestressed concrete pavements (Mogilevich, 1976; Larson et al., 1972; Hargett, 1969, 1970; Zuk, 1972). Most of these articles deal with a 900-foot (275 m) section of experimental prefabricated pavement constructed near Brookings, South Dakota in 1968. This pavement consists of a composite two-layer system of 4½-in. (114 mm) prestressed concrete slabs covered with a 3-in. (76 mm) asphaltic concrete surface course. The slabs rest on a 3-in. (76 mm) minimum crushed gravel base.

Prestressed pavements have been recognized as a possible economic solution to the problem of providing an efficient highway network (Mogilevich, 1976; Christensen et al., 1968; Pasko, 1972). Prestressing can reduce pavement thickness 25 to 40 percent and still offer the same load carrying capacity as unstressed construction. Prestressed pavements are more crack resistant, more enduring, and offer better response to moving loads (Mogilevich, 1976). Because prestressing tends to create a more dense concrete, frost resistance can be increased. Loads are distributed over larger areas in prestressed pavements so that even poor subgrades can adequately support the pavement without costly special preparation. It, therefore, appears that prestressed pavements are at least appealing as a prospective zero-maintenance system. Prefabricated prestressed panels can take this desirability a step further by providing virtually no prestress losses to the subbase that occur with post-tensioned poured-in-place pavements.

The slabs used on the South Dakota project are 24 by 6 ft (7.3 by 1.8 m), containing 10 seven-strand high strength cables (yield stress of 270,000 psi (18,980 kg/cm^2)) prestressed to 405 psi (28.5 kg/cm^2) and placed in the longitudinal direction of the slab. Transverse reinforcement consists of No. 3 deformed bars spaced 1 ft (30.5 cm) apart (Figure 63). The transverse reinforcement is unstressed. Slabs were laid with the long side transverse to traffic flow, and also, in the same direction as traffic flow, each section composing about half of the total 900-foot (275 m) length (see Figure 64). Joints consist of tapered grout keyways (Figure 65) that provide transfer of shear across

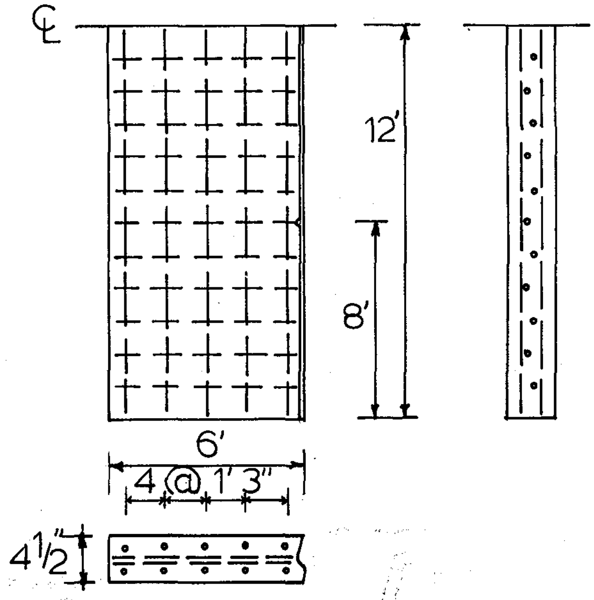


FIGURE 63 TYPICAL PANEL DETAILS, SOUTH DAKOTA PROJECT

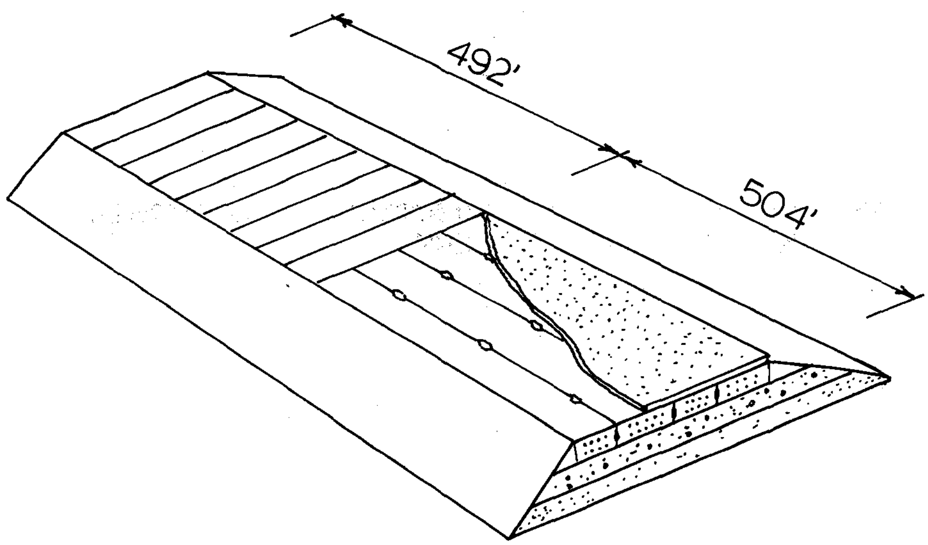


FIGURE 64 ISOMETRIC VIEW OF SOUTH DAKOTA PROJECT TEST SECTION

(1 ft = 0.3048 m)

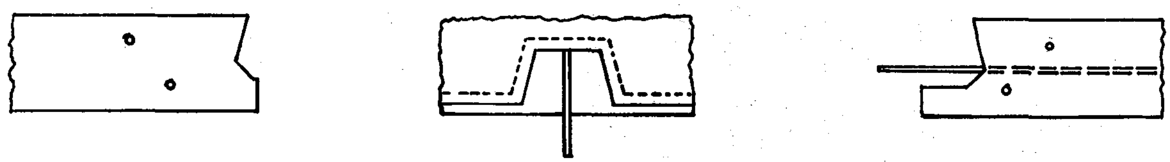


FIGURE 65 GROUT KEY AND CONNECTION JOINT DETAIL, SOUTH DAKOTA PROJECT

the joints but are not strictly rigid joints that also provide moment transfer (Larson et al., 1972). After 4 years of service, only minor reflective cracking was observed during winter months, which tended to reseal in warmer weather. Reflective cracking can be prevented with plastic sheets similar to those described in Highway and Heavy Construction (1976). It is not known which method proved more successful -- longitudinal or transverse laying of the slabs -- but it has been suggested that the number of joints in the longitudinal direction (i.e., transverse joints) be kept to a minimum (Mogilevich, 1976). It has also be suggested to keep joint spacing between 15 and 20 ft (4.6 and 6.1 m) (Nussbaum et al., 1977) to prevent cracking between joints. With prestressed pavements, however, this distance may be increased, evidenced by the behavior of the South Dakota pavement.

Once again it appears that properties of prefabricated prestressed slabs are similar to those of conventional poured and post-tensioned pavements. Similar design processes have been followed with the exception of treatment of joints. Joints will be discussed later in this section.

Prestressed slabs need not be stressed with steel tendons alone. Various synthetic materials can also be used, provided these tendons are anchored properly and are sufficient to carry the load. Many fibers are in existence today that possess much higher strength-to-weight ratios than steel. The consequence is a much lighter slab and perhaps significant savings. Perhaps a combination prestressed concrete and fiber reinforced or polymer impregnated concrete may prove to be a good candidate for zero-maintenance pavements. Indications are that this material would be quite expensive initially but may have great endurance and require little maintenance.

The positive aspects of prestressed concrete far outnumber any shortcomings it may have. The quality control possible with prefabrication only increases the desirability of this type of construction.

4.6.2.3 Other Materials

This section reviews plain concretes, that is, unreinforced materials.

Many studies have shown the economy involved in the use of plain concrete pavements, depending, of course, on availability of material, especially sound aggregate. The question now arises whether or not plain concrete can be used in a prefabricated highway system.

Mogilevich et al. (1976) suggests that slabs of plain concretes must be relatively small to withstand installation stresses. These same slabs must also be relatively thick to withstand traffic loads. The increased number of slabs (and hence joints) involved will increase construction time and labor costs, indicating little, if any, increase in savings. Larger number of joints increase reflective cracking on a per unit area basis (if an asphaltic concrete overlay is used) or may impare pavement smoothness. These factors have a net negative connotation on the desirability of plain concrete slabs and hence they are not generally recommended for use in prefabricated highways (Mogilevich, 1976).

Asphaltic or bituminous concretes are flexible in nature and will thus pose many problems in construction if installation stresses are to be kept to a minimum. They must either be cooled to make the material more rigid, or be kept to very small sizes, which will tend to render them useless. They tend to deform greatly and are not recommended for surface courses. Mogilevich recommends their use as base material, but even in that role they appear deficient.

Composite construction such as that used in the South Dakota project (1967-68) offers yet another alternative for the designer. Composite pavements (i.e., monolithic concrete paved over rigid prefabricated slabs) can provide both a crown to facilitate runoff and also a waterproof friction surface for traffic, eliminating wear of the actual structural concrete. As mentioned previously, reflective cracking can be a problem because of the hinged slabs resting beneath the asphaltic surface course that flexes as traffic passes above. This has to be accounted for and perhaps eliminated, if feasible.

Composite pavements such as this may qualify as zero-maintenance types. Consider the following: after construction, traffic is allowed on the pavement. If adequately designed, the structure may not fail within its first 20 years of service. If the surface course shows signs of excessive wear, it may either be stripped away (allowing examination of the slabs) and any necessary repair or an overlay can be applied directly above the old layer, which will add to the total structure and help to strengthen it for subsequent service.

4.6.3 Joints

This section contains specific information on joints for prefabricated pavements. For more material on joints, see other sections of the report.

It is obvious that when concrete slabs are joined together, the joints are very important in the transference of loads, the prevention of water and foreign substance intrusion, the prevention of differential settlement of the slabs, and their ability to cope with thermal expansion/contraction cycles. There are two basic types of joints: constant width and nonconstant width. Slab/joint configurations can be articulated joints, which permit independent movement of slabs, elasto-pliable joints, which permit limited (usually shear) load transfer, and rigid joints, which tend to make the slab act as a monolithic pavement (Figure 66).

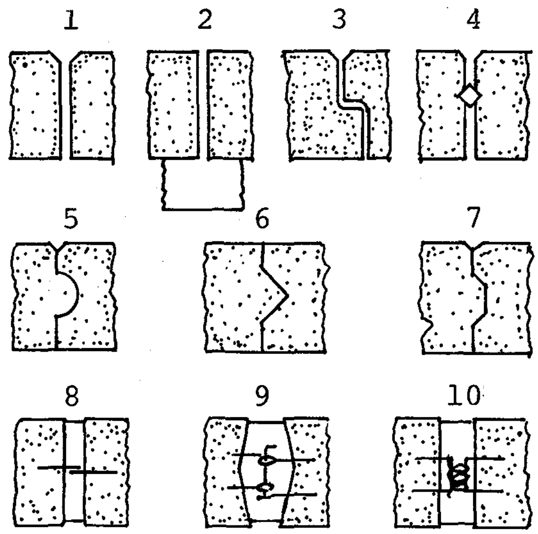


FIGURE 66 TYPICAL JOINT TYPES: 1,2: ALLOW INDEPENDENT SLAB DEFLECTION; 3,4,5,6,7: PROVIDE VARIOUS DEGREES OF SHEAR TRANSFER BETWEEN SLABS; 8,9,10: SEMI-RIGID JOINTS TO TRANSFER SOME FLEXURAL STRESSES BETWEEN SLABS

Nussbaum (1977) concludes in his assessment of concrete pavement improvements that loads should be transferred across joints from slab to slab to help distribute loads over wider areas. He goes on to state that joints should have a depth-to-width ratio not exceeding 3:1 and that joints in the longitudinal direction should be sufficient to hold the slabs in place, prevent faulting, and inhibit water intrusion. With these factors in mind, rigid joints would appear superior to others for permanent pavements. Rigid joints, however, are more complex to construct.

In general, joints in prestressed slabs are of a constant width in order to maintain uniform prestressing and to help prevent prestress losses to the subbase. Some rather novel joints for prestressed pavements include methods of jacking the slabs apart after joining connecting bars, either with mechanical jacks (Figure 67) or by means of a hollow tube inserted in the joint (Figure 68), filled under pressure with a grout that hardens (Mogilevich, 1976). Other joints can include those used to monolithic prestressed pavements such as the method described by Pasko (1972). This pavement was constructed near Washington in 1971 for the "TRANSPO 72" show; the joints were fashioned from 6-in. (152 mm) I-beams imbedded in transition slabs between the major post-tensioned portions of the pavement.

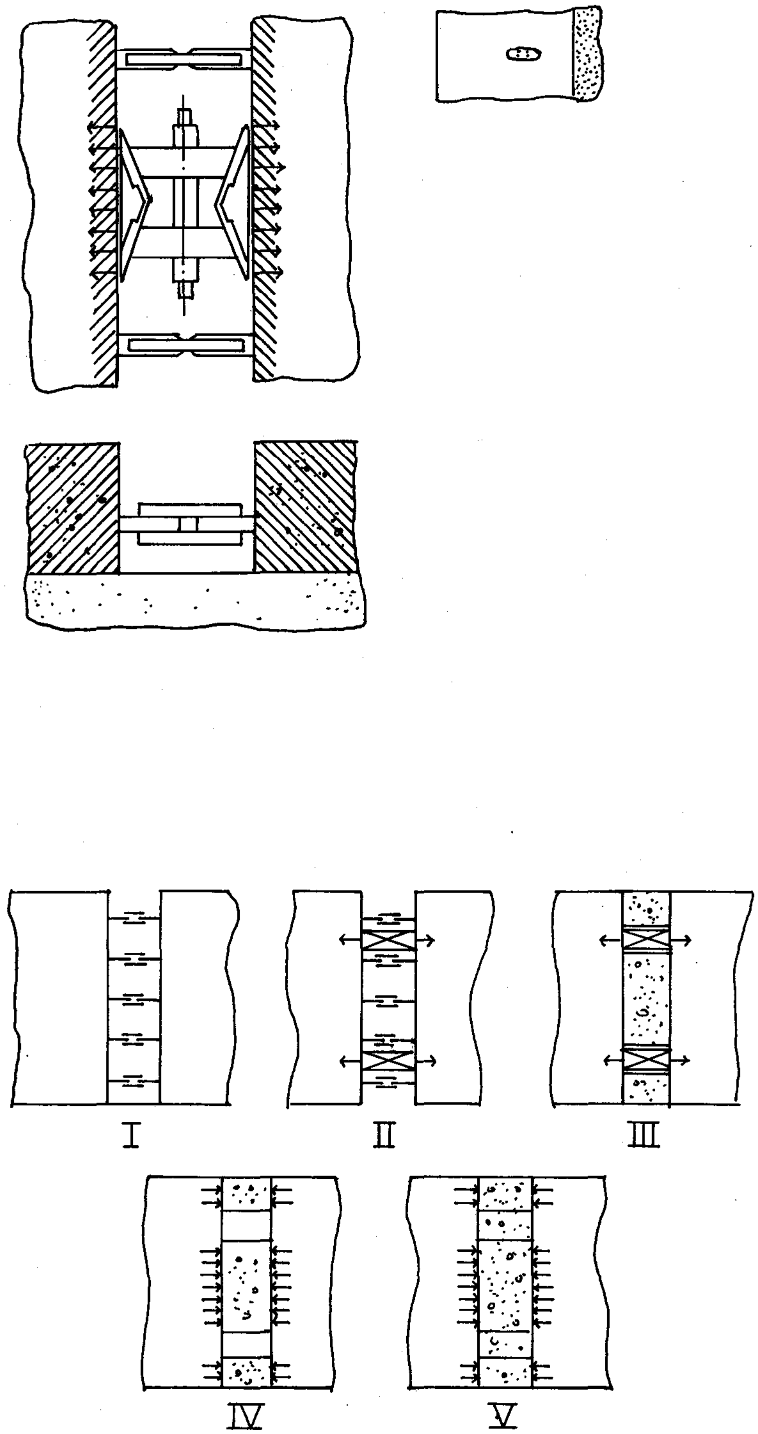
Other types of slab utilize variable width joints that can expand and contract with varying temperature. These types of joints are usually filled with elasto-plastic materials, which do not inhibit contraction of the joint. The elasto-plastic joint filler must adhere well to the edges of the slab to insure that no water will penetrate the joint and initiate pumping action. The joint should be durable in all types of environmental conditions it may encounter.

All pavements (especially those with constant width joints) require expansion and/or contraction joints to combat thermally-induced strains. Standard types in service at the present are compatible or can be adapted for use with prefabricated pavements.

Pavement slabs can be strung together with cables (Figure 69 running through conduits cast within the slabs similar to those described by Zuk (1972). Regardless of the method used, however, joints should be at least as strong and enduring as the slabs they are connecting.

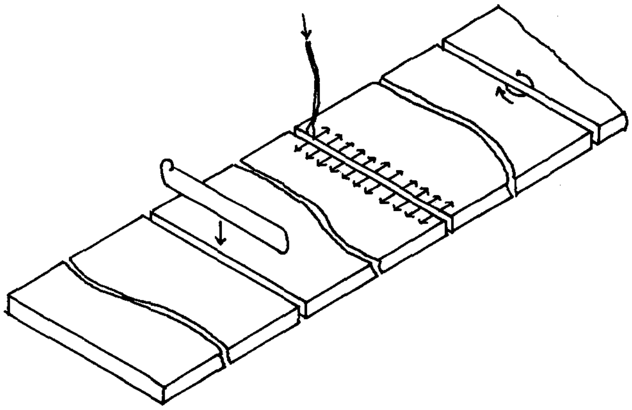
4.6.4 Foundations

A crucial portion of the active pavement system is the supporting structure. The pavement base must not permit excessive sagging under traffic and must be water resistant. The surface of the base must be even to provide full contact with rigid slabs. Crushed gravels, either continuously graded or gap graded, which are at least 3 in. thick, can be satisfactory. Soil cements are



SHOWS CONSTRUCTION SEQUENCE

FIGURE 67 JOINT FOR PRESTRESSED SLABS USING MECHANICAL COMPRESSION



CONSTRUCTION STEPS

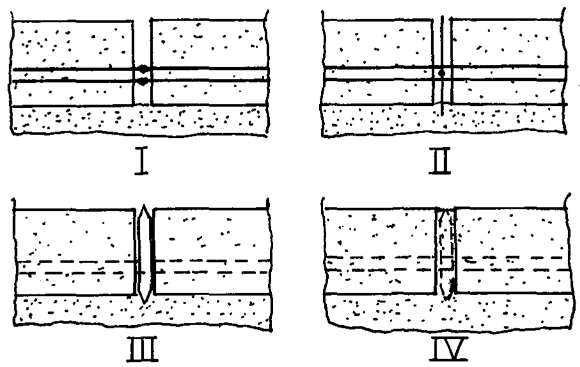


FIGURE 68 JOINT FOR PRESTRESSED SLABS USING HYDRAULIC COMPRESSION

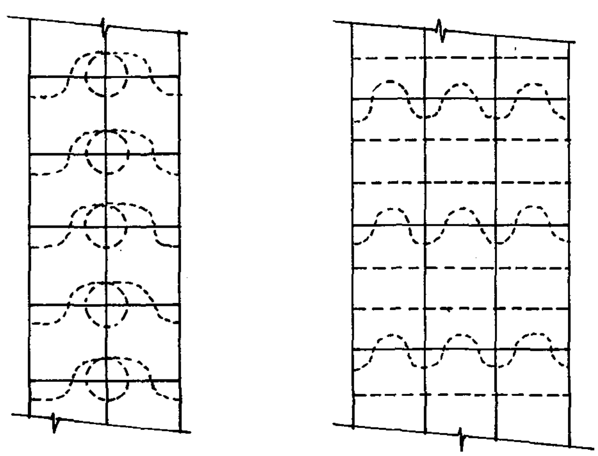


FIGURE 69 PLAN VIEW OF BIAXIAL COMPRESSION JOINING OF PANELS BY PRESTRESSING TENDONS RUNNING THROUGH CONDUITS

also good candidates. Any material selected must have uniform strength and density throughout and must compact uniformly to prevent differential settling. Crushed stone or gravel is considered an all-around good subbase material (Mogilevich, 1976; Child et al., 1963).

Strip footings (Figures 70 through 72 can also be used to support the slabs (Mogilevich, 1976). Piles or isolated footings (Figure 73) are other proposed devices that may prove successful. These types of footings can once again be prefabricated or cast-in-place. The foundation should be vibro-compacted, the slab placed on the base and itself vibrated to insure good contact so that load transfer to the supporting layers is uniform and adequate.

Many solutions are possible, but extensive investigations are required to determine which types of foundations can be applied to specific site conditions.

4.6.5 Manufacturing

Prefabricated slabs can be manufactured in much the same manner as prefabricated building or architectural details are manufactured at the present time. Many facilities are presently available to create slabs that can be stockpiled or shipped throughout the country and used when necessary.

Plants set up especially to manufacture pavement slabs are outlined by Mogilevich et al. (1976). They include vacuum hardening chambers, steam chambers, and conveyor networks that result in assembly line efficiency of operation. With relatively little capital investment, slabs could be mass-produced in this country at costs that can effectively rival conventional poured-in-place pavements.

4.6.6 Construction

The method used in South Dakota (Larson et al., 1972) and in the USSR (Mogilevich, 1976) to lay the pavement slabs involves the use of a motorized crane, either working from the shoulder of the road or on rails running parallel to the road construction (Figure 74). At the present, the latter type of equipment would need to be specially built. However, since rails were used (similar to the rails used by slipform pavers) on the South Dakota project, with apparent success, it seems likely that the use of a rail-mounted gantry crane to pave the slabs may prove more efficient.

The sequence of operations is similar to that of conventional pavements. The slabs must be transported to the site. The subgrade must be prepared, followed by placement of the base and/or foundations. The slabs are then lifted in place and joined together. Finally, the joints are sealed.

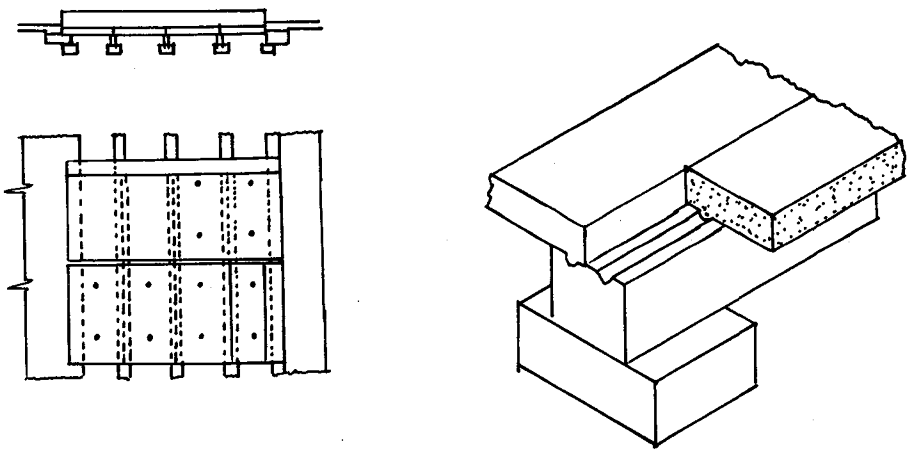


FIGURE 70 PREFABRICATED SLABS ON LONGITUDINAL FOUNDATIONS

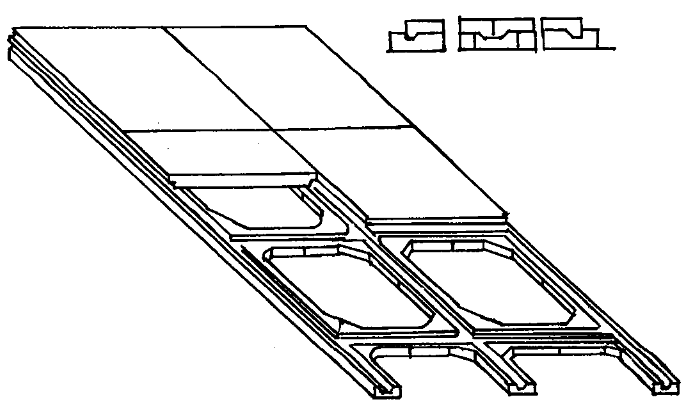


FIGURE 71 PREFABRICATED SLABS ON PREFABRICATED GRID FOUNDATION

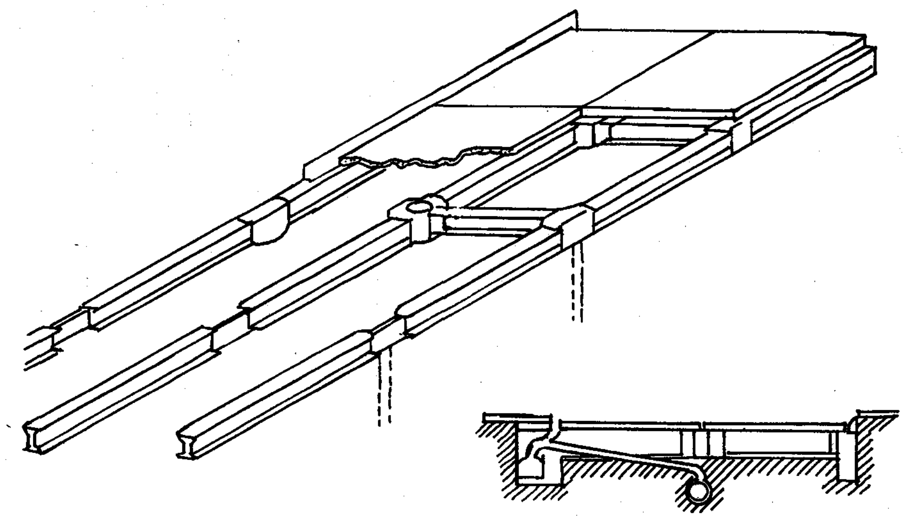


FIGURE 72 PREFABRICATED SLABS FOUNDED ON LONGITUDINAL AND TRANSVERSE BEAMS

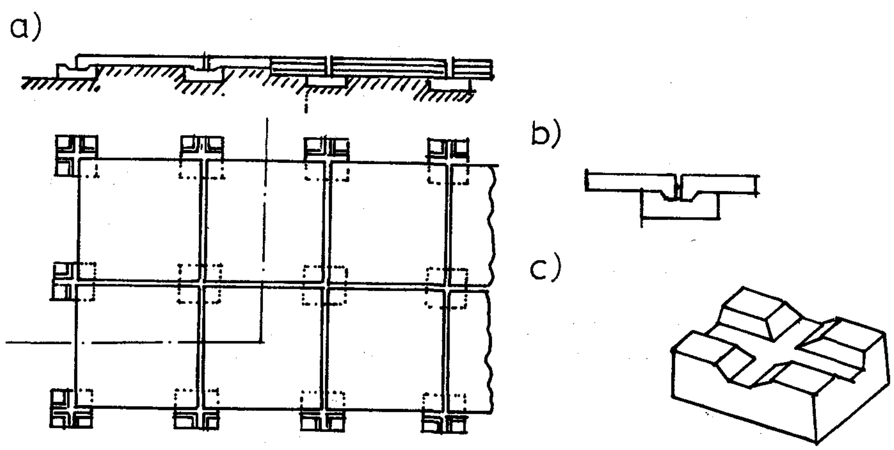
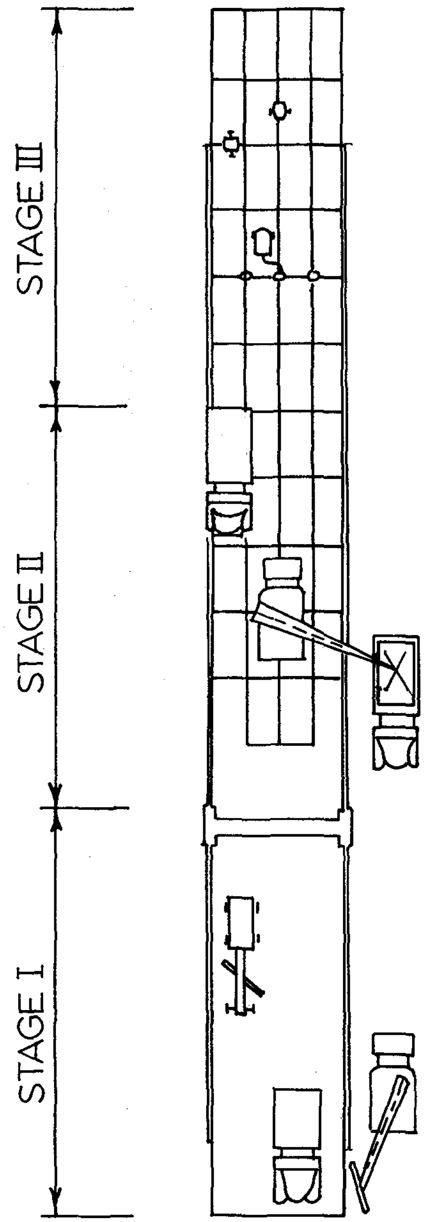


FIGURE 73 DOWELED SLABS LAID ON ISOLATED ANGULAR FOOTINGS

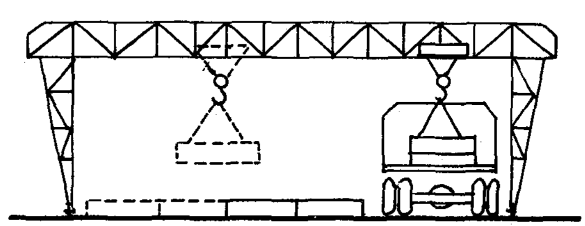
STAGE I: PLACE RAIL/FORM AND PLACE AND PREPARE BASE

STAGE II: PLACE AND VIBRATE SLABS

STAGE III: COMPLETE JOINTS



A CONSTRUCTION WITH MOBILE CRANE



B OVERHEAD GANTRY CRANE CONSTRUCTION

FIGURE 74 CONSTRUCTION METHODS

Cost of the South Dakota project was approximately \$18 per square yard (\$23.7/m^2) (Larson et al., 1972) as compared to \$7 to \$8 for conventional 10-in. (25.4 cm) pavement. With a large-scale operation, costs could drop below \$15 (1968 prices) per square yard. It must be kept in mind that this prefabricated pavement was designed as a heavy-duty pavement, equivalent to approximately 15 in. (38.1 cm) of plain concrete. Initial cost is higher, but maintenance cost during the first 20-year service period may prove far less than for conventional pavements.

4.6.7 Conclusions

After reviewing the various types of prefabricated pavement construction, certain advantages of this type of construction should become obvious. The desirable features cited by Mogilevich (1976) and Žuk (1972) are indicative of the general concensus toward prefabricated pavements available through the rather limited literature. The primary advantages associated with the prefabricated pavements include: 1) better quality control and homogeneity of the various elements with fabrication carried out in factories and proving grounds; 2) a nondestructive method of repairing utilities, etc., that are located beneath a street or road; 3) quicker opening to traffic due to the absence of in-situ curing time; 4) better allotment of labor, machinery, materials and energy; 5) ability to dismantle and reuse the elements; and 6) the possibility of increased durability and classification as a zero-maintenance concept.

Because of the assembly-line nature of prefabrication and the increased cost of raw materials and labor, prefabricated slabs (especially the prestressed variety) may become an economically viable alternative to conventional pavement, especially if the prefabricated pavement is classified as a zero-maintenance type. Also recommended are areas for the best use of prefabricated pavements: 1) city streets with intense traffic and many subsurface utilities; 2) places where rapid construction is required or where detours are impossible or very costly; 3) in extreme weather conditions when conventional paving cannot be used; 4) use on temporary roads or detours where dismantling capabilities can prove economical; and 5) use in places where conventional equipment will not fit (tunnels, etc.). Perhaps the best use of prefabricated pavements can be for multilane high speed highway networks.

Though prefabricated pavements possess a large number of desirable features, some shortcomings are present. Many can easily be overcome, however. The primary disadvantages include: 1) relatively large number of joints that may impair pavement smoothness and increase cost; 2) load transfer between slabs at joints; 3) difficulty of laying standard slabs (especially rectangular) around small radii curves, necessitating the use of nonstandard slabs or monolithic construction; 4) large initial expenditure on special plants and equipment; 5) possible high construction cost; and 6) the required development of special construction machinery to lay the slabs and prepare the supporting structures, if necessary. Many of these problems are also present with conventional pavements, and solutions can be applied to either type of construction after being suitably modified.

The most promising type of prefabricated paving element appears to be a prestressed slab. The material from which it is fashioned can be portland cement concrete, steel or synthetic fiber reinforced concretes, a PCC that has been impregnated with a polymer material, or a combination of the above.

Soil cement base courses, strip footings, keyed sleeper slabs, piles, etc., are good supporting structure candidates.

Composite pavements created from prefabricated rigid slabs covered with either a flexible and waterproof wearing surface of asphaltic concrete, or a continuous rigid concrete surface course also seem likely candidates in the search for effective prefabricated pavements, especially after further performance results of the South Dakota project become available.

Many factors influence the economical design of pavements. Perhaps further investigation of more exotic foundations and joints will prove valuable to the body of knowledge already available. Prefabrication can easily be tied in with these topics to perhaps develop a virtually maintenance-free pavement.

4.7 JOINTS

4.7.1 Introduction

This chapter presents the findings of a study on innovative and unconventional joints, as well as the evaluation of the joint components and systems uncovered. The evaluation is based on existing technical data and/or performance records and is intended to demonstrate whether or not these components and systems are feasible for use in zero-maintenance pavements or warrant further investigation.

A brief summary of the types, functions and distresses of conventional joint systems and components currently used is also presented. The conditions establish the needs to be fulfilled through innovations in joint design and also establish criteria that can be used in designing and evaluating innovative joints.

4.7.2 Background

Relatively little information on innovative joints was found to be in existence; it is believed, however, that a rather exhaustive study was conducted on reference sources readily available. The topic being only one of the contributing type in the total structural system, an all-inclusive search was not justified.

The present search for information on innovative joints included the review of titles and abstracts of publications printed within the past 10 years and contained in one of the following: Engineering Index Annual, National Technical Information Services (NTIS) publications list (last 5 years of references accessed by computer), and the card catalogs at both the Federal Highway Administration Department of Transportation (FHWA-DOT) Library and The John Crerar Library. Additional references and information were obtained through contacts with personnel and faculty of the following institutions and universities: Georgia Department of Transportation, J.B. Thornton; Florida Department of Transportation, Mr. Stelzenmuller; Michigan Department of State Highways, Mr. H. Shaffer; and Cincinnati University, Mr. Cook.

4.7.3 Joints in Rigid Pavements

Joints are an integral part of all types of existing rigid pavement systems and therefore directly affect the performance of these pavements. Different types of pavements incorporate joints to varying degrees and also require joints of different sorts to perform various functions.

Among the different rigid pavement types presently used are jointed plain concrete pavements (JCP), jointed reinforced concrete pavements (JRCP), continuously reinforced concrete pavements (CRCP), and to a much lesser extent, prestressed concrete pavements.

4.7.3.1 Conventional Joints

The types of conventional joints presently used can be grouped according to the categories shown on Figure 75. Transverse joints are incorporated in rigid pavements to provide for the relief of tensile, compressive, and warping stresses. These stresses result from pavement shrinkage, frictional restraint, loadings, temperature changes and moisture gradients. Longitudinal joints are provided to allow for the relief of curling stresses resulting from temperature gradients. Included among the distresses experienced by joints are faulting, raveling, spalling, pumping, blowup, cracking and infiltration of incompressible fines and water. The inadequate performance of load transfer devices (LTD) and joint sealants are responsible for most, if not all, of the above-mentioned distresses. Wagner (1977) provides an excellent summary of the requirements joint components must meet in order to perform successfully. Standards and specifications for materials comprising the joint components, i.e., tie bars, sealants, etc., are also given by Wagner (1977).

Numerous excellent detailed design and construction procedures for conventional joint systems and components are available in NCHRP and PCA reports (1973, 1974, 1975; Carlson, 1974). In dealing with new structural systems for zero-maintenance pavements, one might very well integrate components of proposed and

A. TRANSVERSE JOINTS

<u>Weakened Plane Contraction Joints</u> (Doweled or Plain)

Inserts (Partial or Full Depth)
Formed
Sawed

<u>Elastic Joints</u>

Inserts (Partial or Full Depth)
Formed
Sawed

<u>Expansion Joints</u>

<u>Construction Joints</u> (Doweled, Tied or Plain)

Butt
Keyed

<u>Bridge Approach Joints</u>

Multiple Expansion Joints
Bituminous Concrete Insert
Joint Utilizing Wide Flange on Sleeper Slab
Anchored Lugs

B. LONGITUDINAL JOINTS

<u>Weakened Plane Joints</u>

Lane Joints
Shoulder Joints

<u>Construction Joints</u>

Butt
Keyed

FIGURE 75

JOINT TYPES

existing pavement systems to obtain the desired service life of the system. For example, in studying and experimenting with new types of rigid wearing surfaces, both longitudinal and transverse connections between these pavements might very well consist of joints that have been used in the past and have demonstrated the potential for obtaining a 20-year maintenance-free life. Joints of this type are discussed (in conjunction with the new structural systems in which their utilization is intended) in other chapters of this report.

4.7.3.2 Innovative and Unconventional Joints

Limited information on innovative or unconventional joint types, load transfer devices and sealants, as well as sealant practices, has been obtained. Since this information is well documented elsewhere, all the details incorporated in the evaluation have not been duplicated in this chapter. Instead, summaries with appropriate references to the original works are presented in the following paragraphs. It should also be pointed out that the associated cost expected to be incurred through the utilization or realization of these proposed schemes and concepts has not been considered as desirable or undesirable in the evaluation.

Elastic Joints - Elastic joints have not been used extensively in pavement design in the United States. Elastic joints are a form of tied weakened plane joints that allow for the controlled transverse cracking of continually reinforced concrete pavements. Experience in Sweden (Persson and Friberg, 1969) indicates that when these joints are incorporated in pavement design, the 0.5 and 0.7 percent longitudinal reinforcing steel can be reduced to as low as 0.2 percent and still provide comparable performance under virtually equivalent operational and climatic conditions.

A high percentage of longitudinal steel is considered unnecessary since its early age objective to promote the formation of minute random cracks, through tensile stress transfer, is partially replaced by the inclusion of these elastic joints. Also the longitudinal steel's continuing function has been found to be insignificant after the crack formation; crack widths at different temperatures in mature pavements appear to be directly related to crack spacing and temperature and not to the amount of reinforcing steel. In this manner, a truly continuous concrete pavement can be formed; a saving on steel, as well as labor and material costs required to construct conventional contraction joints, is also realized.

Elastic joints are formed by first coating the working steel for a length of approximately 3 ft (1 m) on either side of the weakened plane location with a steel-to-concrete bond breaker such as cutback asphalt, and later by providing a crack inducer either above or below the desired location of the weakened plane joint according to conventional methods. The joint may or may not be sealed depending on the design, operating and climatic conditions characteristic to a particular pavement system (discussed in more detail in a subsequent subsection).

The 360-foot (110 m) long by 26-foot (8 m) wide and 6.3-in. (16 cm) thick experimental section of the Swedish divided highway with joint spacings from approximately 14 to 20 ft (4 to 6 m) was found to be in excellent condition after years of service (Persson and Friberg, 1964). Observations included that the 0.2 percent reinforcing steel was apparently sufficient to provide joint movement and adequate load transfer. The experimental pavement was subjected to air temperature changes of approximately 88 °F (31 °C) and heavy traffic with a large percentage of trucks.

The 3-year performance record is not enough to permit the evaluation of the proposed system for use in zero-maintenance pavements. It is recommended to obtain and evaluate performance records in the latter phases of this investigation for the experimental pavement constructed in Sweden and any other performance records of similar road types. An additional 10 years of service have been provided by the Swedish road since the publication by Persson et al., 1969.

The concept of incorporating elastic joints and in turn reducing the percent of longitudinal reinforcing steel does not have merit and shows potential for providing the required service life in zero-maintenance pavements. The degree of steel reduction outlined above may not be feasible in all pavement applications and must be investigated further.

Rigid Joints - Various types of rigid joints (actually connections) for rigid concrete pavements constructed from prefabricated slabs have been proposed and also utilized by Russian engineers (Mogilevich, 1976). These rigid joints allow for no relative movement between adjacent slabs and fall into the category of either unstressed or prestressed connections. The applicability of prestressing rigid joints depends on the type of prefabricated pavement connected. Russian engineers recommend that connections for prestressed, prefabricated slabs should also be prestressed in order to provide an even more rigid and durable, as well as impermeable, connection. The performance of these types of connections in either experimental or field applications was not quantitatively described (Mogilevich, 1976). Numerous difficulties, such as fabrication and installation techniques, are reported in a majority of the concepts presented. Materials or their properties are not reported as presenting problems in transforming these concepts into operational designs. The movement experienced by the rigid pavement composed of these prefabricated slabs is accommodated by expansion joints with a spacing from 40 to 100 meters (dependent on the type of rigid pavement) and in some cases with contraction joints. These articulated joints may or may not consist of the conventional type of joint utilized in the United States. Nonconventional articulated joints are presented in the next section under types of joints.

The technique outlined by the Russian engineers consists of the following basic steps in constructing rigid joints:

1. Placing the prefabricated slabs at predetermined locations and spacings;

2. Removing any undesirable objects such as dirt from the gaps;

3. If present, connecting the working steel that was left protruding from the prefabricated slabs by either welding or joining with attachable mechanical devices;

4. If desired, stressing the joint by exerting pressure against the vertical sides of adjacent slabs;

5. Filling the gap not occupied by the stress inducing device with quick-setting grout;

6. Removing the stress inducing device after the concrete has set and filling the remaining voids with additional grout; and

7. Sealing any additional voids formed during the construction process with conventional (i.e., hot and cold poured asphaltic compounds) sealants.

Some variations to the above steps do exist, depending on the type of concept utilized. Various concepts are presented below. Specific design and construction details are not presented. These details, however, will be presented in the forthcoming design manual for the concepts showing the most potential for providing a zero-maintenance pavement.

A. Unstressed Rigid Joint Concepts

1. The support of slab corners on embedded prefabricated angular foundations is shown on Figure 73. Dowels are installed between the slabs to increase the rigidity of the connection. A cement grout is used to fill the gaps between the slabs. It is noted that the angular foundations (i.e., sleeper slabs) provide restraint against horizontal movement of the pavement by acting as anchoring devices:

   a. A prefabricated reinforced concrete framework could also be utilized to provide support to the prefabricated slabs as illustrated on Figure 71;

b. Figure 72 illustrates a derivative of the concept shown on Figure 71. The framework supporting the prefabricated slabs consists of longitudinal and transverse beams. This concept, however, was put to design and resulted in the construction of a rigid pavement in Switzerland, consisting of 11 by 23 ft (3.5 by 7.0 m) prefabricated slabs;

2. Other types of concepts for connecting prefabricated slabs have been presented in previous sections and Figures 76 , 77 and 78. In all cases, either the reinforcing steel of the prefabricated slab or embedded hoops extending from the sides of the prefabricated slabs serve to form the primary connection between the slabs. All joints are again filled with a cement grout after the steel has been spliced or connected:

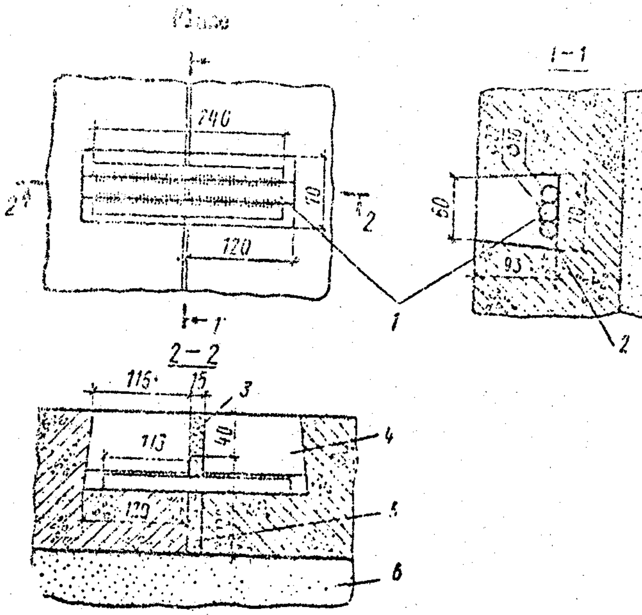
    a. Figure 76a shows the connection to consist of the butt type. Additional rods are utilized in recesses along square edges of the slabs to obtain the necessary weld strength. A mechanism such as the one shown on Figure 76b may be used instead of the weld to connect the reinforcing steel;

    b. Figure 77 illustrates a transverse connection similar to the one shown on Figure 76. This concept, however, provides for the incorporation of additional reinforcing to be located in a space left between the adjacent slabs, thus forming a "reinforcing beam";

    c. Figure 78 illustrates a concept applicable for connecting prefabricated slabs manufactured with chamfered corners. Embedded hoops extending from the vertical sides of these corners are joined with wire prior to filling the square void with cement grout. A sleeper slab shown on the figure for providing extra support to the slab edges and rigid joints may or may not be incorporated in the design. The hoops used in the connection process may also be used for lifting and positioning the slabs. Spaces between the square sides of the prefabricated slabs are filled with conventional sealants.

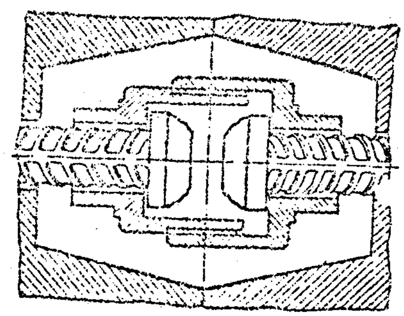
B. Prestressed Rigid Joint Concepts

Four concepts are proposed for stressing rigid joints before they are subjected to operational loads:



A CONNECTION BY WELDING:
1. WORKING REINFORCEMENT
2. ELECTRICAL WELDING
3. "IZOL" CEMENT
4. FILLING WITH EPOXIDE COMPOUND
5. CEMENT SOLUTION
6. SAND/BITUMEN MIX

(ALL DIMENSIONS IN mm)



B COUPLING THREADED CONNECTION OF PREFABRICATED SLABS

FIGURE 76 CONNECTION OF THE SLABS BY REINFORCEMENT OUTLETS

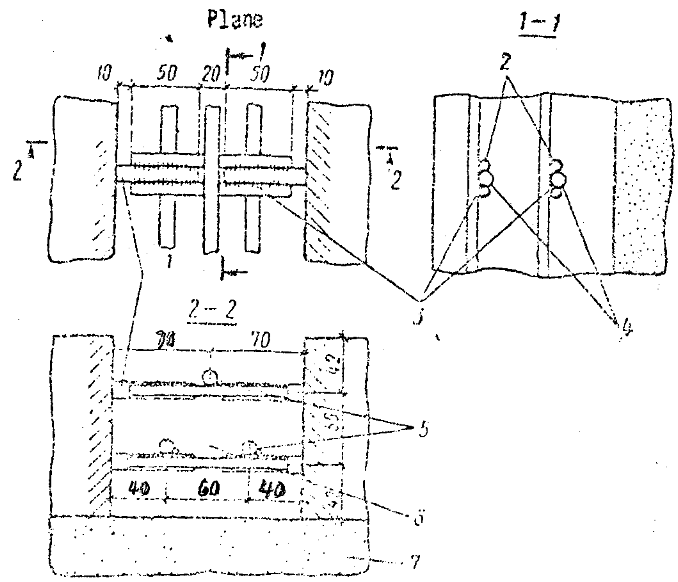


FIGURE 77 CONNECTION OF THE SLABS ACCORDING TO THE TYPE OF "REINFORCING BEAM"

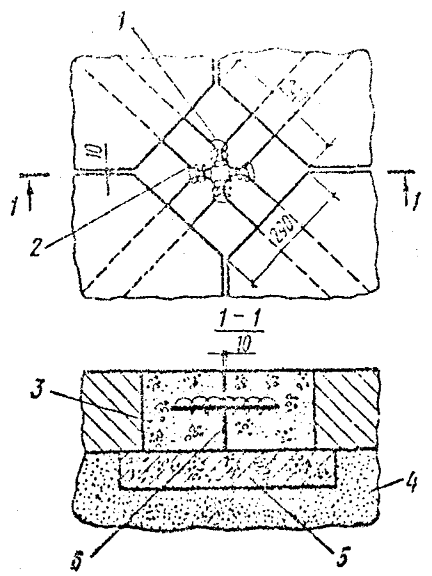


FIGURE 78 CONNECTION OF THE SLAB WITH THE AID OF REINFORCING LOOPS: 1: CONNECTED BY A WIRE; 2: REINFORCING LOOPS; 3: FILLING THE RECESSES WITH CONCRETE; 4: SANDY BASE; 5: STANDARD SLABS 50 x 50 x 8 cm; 6: POURING OF THE JUNCTIONS WITH CLAY-BITUMEN PASTE

1. Post-tensioning of wire strands passed through conduits or channels in the prefabricated slabs after the joint filler has cured is proposed as a solution. Technological advances are still necessary to solve the difficulties encountered in constructing the slabs and installing the pavements (forming of the slabs, installation of strands, application of the stress, etc.);

2. Jacks inserted in a transverse gap left between adjacent slabs are used to push the slabs apart after the working reinforcing steel protruding from the slabs has been joined. This is illustrated on Figure 67. After the gap has been filled with grout and allowed to cure, the jacks are removed, thus putting the joint in a state of compression. The remaining steps in constructing this joint are as outlined above;

3. The mechanical device illustrated on Figure 76b can also be used to put the rigid joint under compression. By leaving recesses in the prefabricated slabs, the mechanism can be installed on the protruding steel reinforcing bars and tightened by screwing the coupling ends together, resulting in the application of a stress across the joint. Prior to stress application, the slabs can be placed in direct contact with one another or placed such that a concrete grout can be introduced between the slab faces. Naturally, this grout is allowed to cure before tightening the mechanism;

4. Figure 78 illustrates a method in which the cement grout introduced into the joint serves a dual purpose. The grout is used to fill the gap and also used to apply pressure by injecting it into a cellular membrane placed between the vertical faces of adjoining prefabricated slabs. The membrane may be positioned to be flush with the pavement surface or allowed to protrude above the pavement surface. For the latter case, the grout-filled membrane must be cut off flush with the pavement surface after the grout has cured.

An evaluation as to which of the above-mentioned joints is better suited for use with prefabricated pavements in obtaining a maintenance-free pavement system is difficult at this time, since most of these are in the conceptual stage. Also, data on the performance of the joint types presently used is not presented by Mogilevich (1976). This limitation necessitates one to make an evaluation based on the recommendations given in the reference, and also on the expected difficulties due to technological shortcomings. Rigid joint concepts that appear to be least desirable for further investigation are those discussed above in A1a, A1b and B1. The fact that the concepts discussed in subsections A2a, A2c and B2 appear to be the least involved regarding construction makes them the most attractive for further investigation. However, if economics is not to be considered and the performance is rated as being the top priority, then all of the concepts proposed should be investigated in more detail in the subsequent phases of this project.

Articulated Joints - Only one unconventional articulated joint was found in the literature (Mogilevich, 1976). The joint is intended to function as an expansion joint and is illustrated on Figure 79. The entire joint, as well as the edges of the two adjoining slabs, is underlain by a sleeper slab. The joint itself is composed of a rubber that is reinforced with steel strips. The end steel strips are rigidly connected to the slabs to prevent a separation between the reinforced rubber and the slabs.

Design details for the joint are not presented. Whether or not the joint has ever been constructed and put into service is also not mentioned. The compressability and extensibility characteristics of this type of joint under varying environmental conditions must be determined experimentally. Overcompression or extension of the joint will reduce the riding quality of the pavement system. It is anticipated that a complex interaction exists between the steel and rubber and therefore will require careful analysis before a suitable configuration is obtained for testing. In light of the above, this concept does not suggest much potential for use in the immediate or near future.

Load Transfer Devices - Ball and Childs (1975) present the details of a laboratory investigation of various conventional and innovative load transfer devices (LTD) for use in rigid concrete pavements. The types of LTD tested are illustrated on Figure 80. Concrete beams and slabs incorporating these LTD were subjected to static and repetitive loads, respectively. The controlled conditions under which all tests were conducted, as well as the performance criteria utilized, resulted in a meaningful and representative baseline that was used to indicate the relative performance of all LTD tested.

The most relevant findings of this investigation are outlined below:

1. Sharp trapezoidal key fillets should be avoided to increase their durability, as was indicated by the longevity of rounded keys;

2. Double rounded or triangular keys, as well as multiple trapezoidal keys, proved to be inferior to single keys;

3. Bound and elastic (anchored and coated) steel ties in conjunction with aggregate interlock were effective and durable. The location and amount of bound tie steel is directly related to performance of closed joints. Little distinction was noted in the ultimate capacity of elastically tied joints for steel percentages of 0.56 percent and lower;

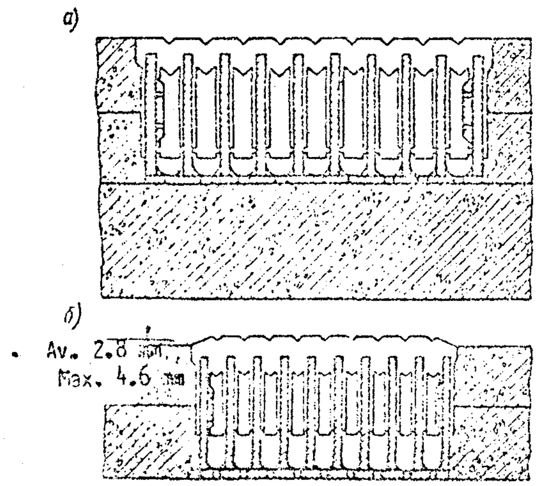


FIGURE 79 FILLING THE JUNCTIONS OF PRESTRESSED PREFABRICATED MONOLITH PAVEMENTS WITH RUBBER, REINFORCED WITH STEEL STRIPS: a) DESIGN OF THE JUNCTION
b) JUNCTION AFTER TEMPERATURE COMPRESSION DEFORMATION

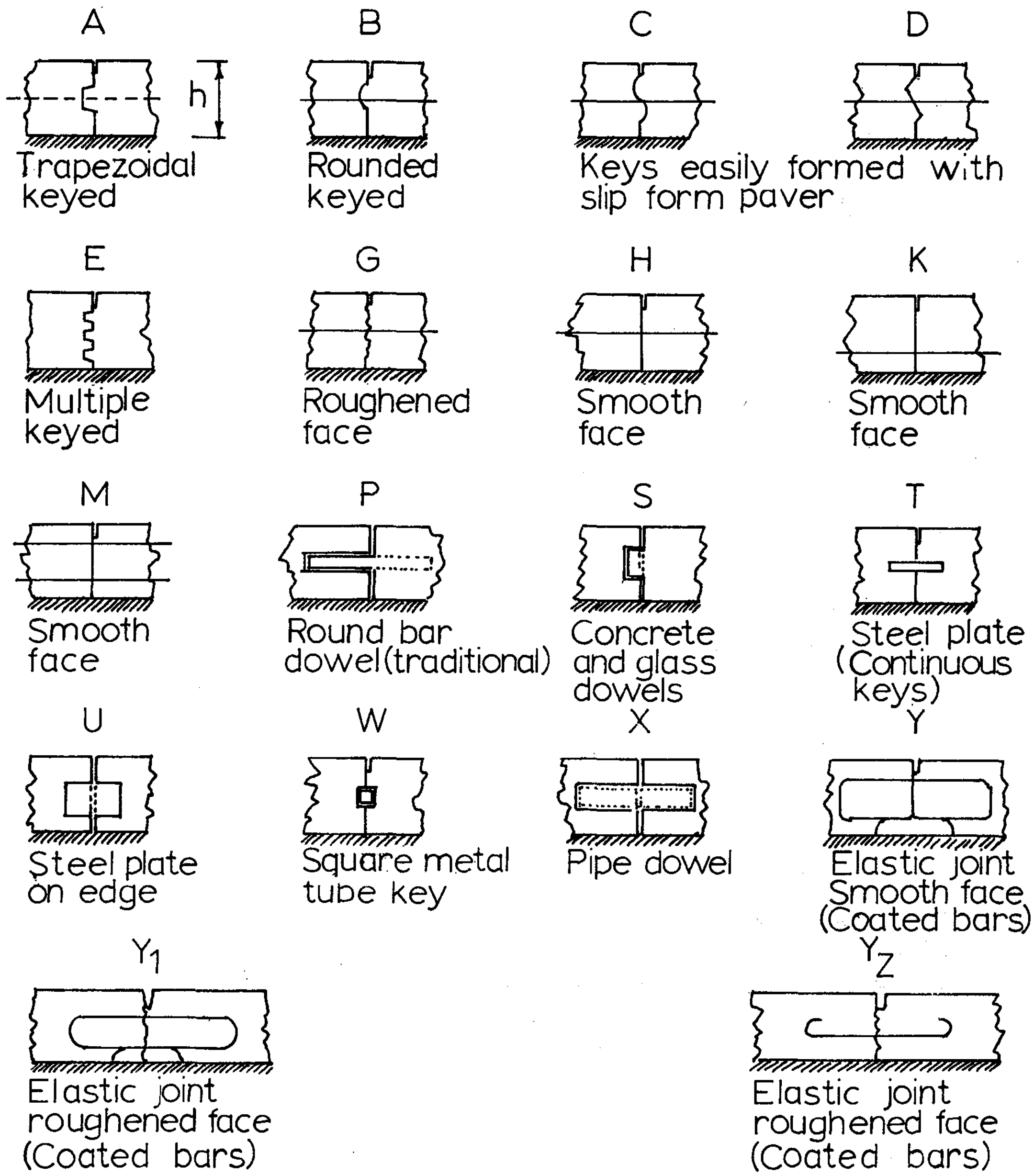


FIGURE 80 JOINT TYPES TESTED

4. Nonmetallic (concrete and glass) dowels were ineffective;

5. Flat steel plates were inadequate to provide load transfer. Vertical plates lacked bearing area and horizontal plates lacked bending resistance;

6. The square tube tested had insufficient shearing resistance;

7. Solid dowels were as good, if not better than pipe dowels due to larger effective concrete areas resulting from the smaller yet strength equivalent solid dowels;

8. Cement stabilized bases improved the performance of all LTD but had the largest effect on improving the performance of aggregate interlock joints.

Scaling effects are not evaluated in this investigation, and it is noted that the field performance of these joint types may be altogether different (either better or worse) than the indicated performance under controlled laboratory conditions. Nevertheless, a good indication of expected field behavior is obtained for the various LTD tested. Minor modifications are suggested and should be incorporated in design to improve the performance of elastic joints. The joints, as mentioned previously, are particularly promising in continually reinforced concrete pavement applications.

4.7.4 Sealants and Sealing Practices

Silicone Seals - Silicone seals have been used in building construction for several years and are now promoted for use as joint sealants in rigid concrete pavements by Mr. J.B. Thornton of the Georgia Department of Transportation.

Low modulus silicones, as for example the Dow 790 silicone, appear to have a definite potential in rigid pavement applications for providing long lasting maintenance-free sealants. Some of the attributes of the silicone, as well as some of its shortcomings, are listed below:

1. The low modulus prevents overtaxing the sealant-to-concrete bond strength, and thus helps to prevent adhesion failures;

2. The lack of shrinkage upon curing prevents the formation of initial tensile stresses that might damage conventional hot and cold poured sealants;

3. Unlike preformed sealants, the lack of joint uniformity or evenness is not detrimental to sealant performance;

4. Installation is enhanced since no mixing agents are required;

5. Protection must be provided for a relatively short time after installation to prevent the intrusion of debris into the uncured silicone; and

6. Potentially higher initial costs, as compared to other hand-tooled sealants, appear to be outweighed by better performance and longer life service.

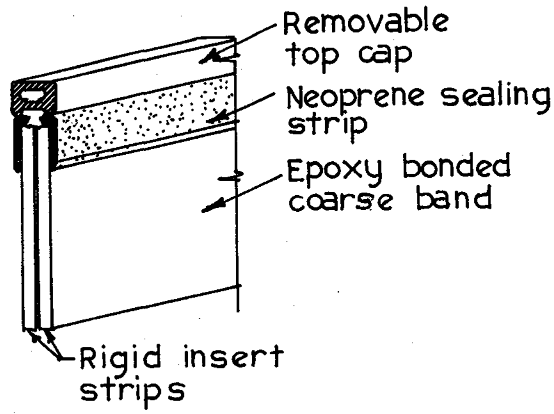
Prefabricated Seals - A prefabricated insert proposed for use in rigid concrete pavement joints is illustrated on Figure 81. The device, when installed, initiates the formation of transverse weakened plane joints and simultaneously seals the joint through the use of a neoprene cap. It is reported that the device has been tested and approved for use on U.S. Navy airfields. The proper installation of the insert is essential for good performance.

Problems to which this device might be susceptible, as experienced by other similar devices, are loss of the epoxy bond between the coarse sand and insert strips, neoprene failure due to weathering or overextension and entrapment of incompressables in the recess formed during wide joint openings. Existing pavements incorporating these inserts should be evaluated with respect to the joint performance.

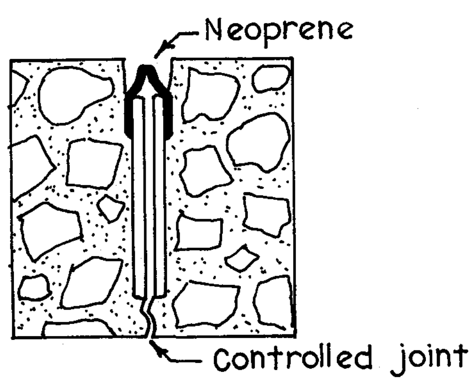
4.7.4.1 Omission of Sealants

Several highways and major roads utilizing rigid pavements of several types have been constructed on stabilized bases in Austria and have performed well for a period of up to 10 years without the "benefit" of transverse and longitudinal joint sealants (Stege, 1976). Joints that are not sealed are sawed into the concrete pavement. The width of the cuts is less than conventional sealed cuts and is approximately 0.12 in. (3 mm). The lack of adequate and long-lasting sealants as well as economic considerations and gain in riding comfort has brought about this trend. At present, however, sealed joints must be provided at shoulder joints regardless of the base type, for pavements on unstabilized bases, for bridge pavements and for pavements located in water conservation districts. It is noted that the success of this practice is directly related to the limited and uniform joint movement. Where joint openings become larger than approximately 0.24 in. (6 mm), it is recommended to construct a conventionally sealed saw cut joint.

Unsealed joints appear to be best suited for continually reinforced concrete pavements supported on stabilized bases due to the relatively small and uniform joint openings characteristic of this type of pavement.



A THE INSERT, DESIGNED TO BOND TO THE CONCRETE, IS PROVIDED WITH A RIGID CAP TO BE REMOVED AFTER THE CONCRETE HAS HARDENED



B INSERT HAS BEEN PLACED IN CONCRETE, CAP HAS BEEN REMOVED, CONCRETE HAS CRACKED BELOW INSERT, AND NEOPRENE SEAL HAS DEFORMED SLIGHTLY TO ACCOMMODATE THE LARGER JOINT OPENING

FIGURE 81 PREFABRICATED JOINT INSERT

4.7.5 Conclusions and Recommendations

A limited number of concepts and unconventional designs for joints demonstrating potential for use in zero-maintenance pavement systems have been presented. Continued investigation is still necessary, however, to develop these concepts into optimal designs that will conform to the criteria and specifications of the pavement systems for which they are intended. Investigative efforts should be focused on both the design and construction of joints since an excellent design improperly constructed or installed can render the joint useless.

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CHAPTER 5

NEW CONCEPTS

5.1 INTRODUCTION

A comprehensive survey of available innovative concepts has been made, the results of which appear earlier in this report. In this survey, in addition to locating the concepts, related topics and problem areas in pavement design and failure were examined. The primary criteria for selection of the promising structural systems for "zero-maintenance" was then reaffirmed, allowing a basis for developing new concepts. The new concepts presented herein are intuitively sound but still in an embryonic stage and will require further analysis (which will be performed in the second stage of the project). It may be noted, however, that some features of the engineering development after the outburst of the energy crisis have changed existing prognostications and any economic notions or considerations engineers had in the past. Therefore, final judgments will have to be made not only from the uncertain cost estimates, but also from the point of view of availability of a material. These aspects will involve considerable and penetrating studies that are not, however, a part of this study. Nevertheless, the assessment of present and future energy and materials resources measured against population growth rates and the demands on the well being of society have established requirements with respect to the capacity and capability of civil engineering. These requirements cannot be met with conventional knowledge and techniques, and hence a researching eye toward new concepts is necessary. Therefore, the next section presents the criteria used for selecting the new concepts.

5.2 CRITERION

Pavements are extremely complex physical systems, involving a complex soil structure interaction problem with a series of variables. Ultimately, all structures have to rest on the in-situ soil -- called the subgrade in pavement design. The engineer placing a structure in or on soil must answer the following basic questions:

1. Will the construction of pavement lead to instability because of a shear failure of the subgrade, either during or after construction?

2. Even if the strength of subgrade be enough to prevent any gross shear failure, will the construction of a pavement (and any embankments in connection with it) lead to excessive deformation in the subgrade?

In engineering terminology, the above two problems are the traditional "stability" and "deformation" problems (Ladd, 1971).

As discussed in Chapter 2, techniques are now available to improve problematical soils from the point of view of stability and deformation. The fact that is not well taken by many is that even though once improved (by stabilizing or any other strengthening techniques), the soils continually face environmental changes. So an ideal subgrade is one that is good not only when construction started but that can maintain its design properties during the design life of the "zero-maintenance" pavement. The need to stress the adequacy of a good subgrade and to maintain the permanency of its properties still remains because many structural engineers tend to ignore the subgrade and concentrate on the superstructure only. We repeat that if an optimum quality subgrade can be achieved and its properties maintained, even some of the conventional methods may provide a zero-maintenance system.

Stresses are transferred to the subgrade soil through base and/or subbase. Needless to say, a proper initial selection, design and maintenance of the properties of the base and subbase during the life span of the pavement system is required. Table 23 is a general format of criteria used to examine every component of a pavement system. In addition to those presented in Table 23 there are other causes and mechanisms of rigid pavement failure. Recently, Nussbaum and Lokken (1977) completed a survey of pavements in the United States and listed several areas of greatest potential for pavement improvement. They are as follows:

1. Loads should be transferred at transverse joints through adequate dowels;

2. Cracking of slabs between joints can be prevented with short joint spacing, 15 to 20 ft; (4.5 to 6.0 m);

3. The joint depth-to-width ratio should be kept under 3:1;

4. Tiebars along longitudinal joints help hold the slab in contact, help prevent faulting, and inhibit water intrusion;

5. Using concrete shoulders creates a strong structural system, reduces water intrusion at the edge of pavement, and improves cross-surface drainage characteristics;

6. Pavement deflections and migrations of fines can be reduced through the use of a stabilized base course. Subbases of this type are strong and more erosion resistant;

TABLE 23

PROPERTIES TO BE EXAMINED (CRITERIA)

A. STRENGTH

Compressive, tensile
Flexural
Dynamic (Impact, Fatigue)
Post fatigue

B. GENERAL -- OTHER PROPERTIES

Creep
Durability
Abrasion resistance
Toughness
Permeability

C. ENVIRONMENTAL EFFECTS

Salt water corrosion
Thermal conductivity
Effect of freeze-thaw
Erosion of base and/or sub-base and subgrade

D. DRAINAGE

7. Increased pavement thickness (stiffness) reduces deflections and improves performance.

Suggestions mentioned above will be incorporated in the proposed structural system wherever feasible.

5.3 PROPOSED SYSTEMS

The following section introduces various concepts that are either new or are modifications of previous systems. The systems adhere to the following basic subdivisions:

1. Systems that improve the conventional system so as to strengthen each subsystem to maintain its improved strength against environmental hazards;

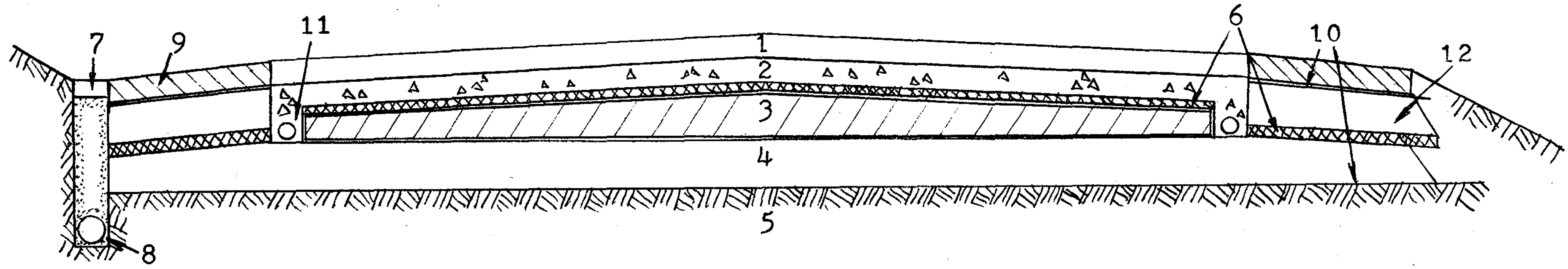
2. Systems in which the stress distribution and load transfer mechanism to the subgrade are modified by variable thickness, anchored slab or anchored base or subbase, etc.;

3. Systems in which the forcing function (load due to the vehicle) and load transfer mechan[illegible] to the subgrade are both changed.

The systems proposed are in a conceptual stage; detailed investigation as to their feasibility and actual stress patterns, in relation to the intuitive thinking, still has to be undertaken.

5.3.1 Improved Conventional Systems

Yoder, Cedergren and others have stressed many times that pavements are subject to failure from the detrimental effects of water penetrating through the surface of the pavements or through a rise in the ground water table. As such, it is necessary to incorporate a system of drains that should convey water away from the structural system. Figure 82 presents such a concept. Drainage trenches are cut along the edges of a pavement and a porous pipe (or a French drain) and collector system is installed. The soil is then strengthened (if required) by surcharging or any other technique detailed in Chapter 2. If the permeability of the soil strata is low, cross draining will also be necessary. The strengthened subgrade must extend under the pavement and shoulders (shown as 4 on Figure 82). Over the strengthened subgrade, a membrane encapsulated subbase is prepared, which will be overlain by a base course (for instance, open-graded bituminous aggregate mixture, OGBAM). In between the OGBAM base and encapsulated subbase, a layer of styrofoam is provided to insulate the base from the subbase. It may be noted that,



1 Surface course
2 Porous base course or asphalt concrete (AC)
3 Membrane encapsulated subbase
4 Prepared subgrade
5 Natural subgrade
6 Thermal insulation
7 Drainage ditch
8 Underdrain (to lower ground water)
9 Paved shoulder
10 Cut-off membrane or filter fabric
11 Base drain (not required with AC base)
12 Shoulder base

FIGURE 82 IMPROVED CONVENTIONAL PAVEMENT WITH THERMAL INSULATION if a porous base is used, provision for drains (shown as 11 on Figure 82) will be made; alternatively, if a base of material like asphaltic concrete or econocrete is used, drains are not necessary. With the utilization of a porous base and/or econocrete, it will be rational to use either a polythene sheet or asphalt-rubber membrane in between the base and surface layer to avoid any friction and thereby avoid reflection cracks. With the subgrade, subbase and base constructed in this manner, a good pavement can be made with a surface coarse grade of any one of the following:

1. Asphaltic concrete overlain by an asphalt-rubber stress-absorbing membrane;

2. Jointed portland cement concrete with the top few inches being sulfur or polymer impregnated concrete (SIC or PIC) or polymer portland cement concrete or an asphalt-rubber stress-absorbing membrane;

3. Fibrous steel concrete with the top few inches being treated as suggested in Item 2.

When the subgrade and base are strong enough and the aggregate used in the asphaltic concrete is of high quality, it is possible to construct a pavement structure so that loads are transferred to the lower layers and can be effectively sustained. Pavements where the subgrade and base were made of broken pieces of trap rock (igneous bassaltic rocks) and the aggregate used in the asphaltic concrete was also from the same material have lasted over 10 years. Rutting over a 10-year period of traffic flow cannot be avoided and has to be corrected.

It is recommended to overlay the asphaltic concrete pavement by a stress-absorbing membrane. It is stated that the tensile strength of the pavement structure is insufficient in responding without fracture to the forces acting on the pavement (Morris and McDonald, 1976). The limited use of elastomers in an asphalt (or asphaltic concrete even) modifies the characteristics in terms of reducing the effects of temperature change on the stiffness and elasticity of the overall structure (Olsen, 1973). The use of asphalt-rubber stress-absorbing membrane has now been demonstrated in the following three forms (Morris et al., 1971):

1. As an elastomer;

2. As an impervious membrane;

3. As a controller of reflective cracking.

Similarly, a properly designed jointed PCC surface with a water-sealer at the top of the surface will function equally well. The fibrous steel concrete, if utilized, will require less thickness than the conventional PCC slab. The concrete slabs, due to their rigidity, are more effective in distributing the load to a larger area, and hence they do not face the problem of rutting. If the infiltration of saltwater is prevented by sulfur impregnation or any other techniques, the steel fibrous concrete is a good candidate. With the steel fibrous concrete, joints of the type shown on Figure 79 are suggested.

For the shoulder area, styrofoam insulation can be placed directly over the strengthened subgrade and the shoulder base and surface can be installed. Shoulder base can be crushed stone, econocrete, open-graded bituminous aggregate mixture, etc. The shoulder surface must be paved using any material like econocrete, porous concrete, or asphaltic concrete pavement. It is now recognized that the shoulders are to provide strength and confinement and need not have the same structural strength as the main pavement. However, between the shoulder surface and shoulder base, a filter fabric should be installed to stop migration of fines into the base course, if the base is porous like crushed stone or OGBAM.

With minor modification, the system can be used in cut as well as fill. The thicknesses of individual layers can be designed using the available design procedures and a knowledge of the properties of individual components.

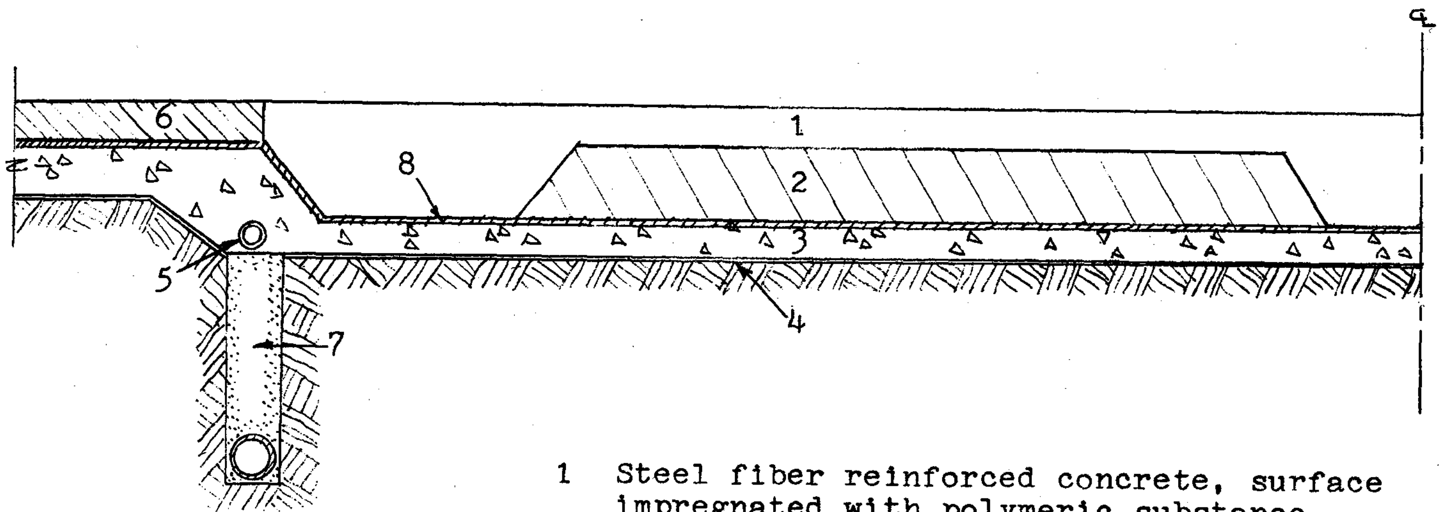
5.3.2 Systems Based on Changing Stress-Distribution Patterns

Two types of systems are proposed: one that requires the strengthening of the subgrade as detailed in Chapter 2 (and used in the system of Section 5.3.1), and the other requiring very minor preparation of the subgrade.

5.3.2.1 Systems Requiring Subgrade Preparation

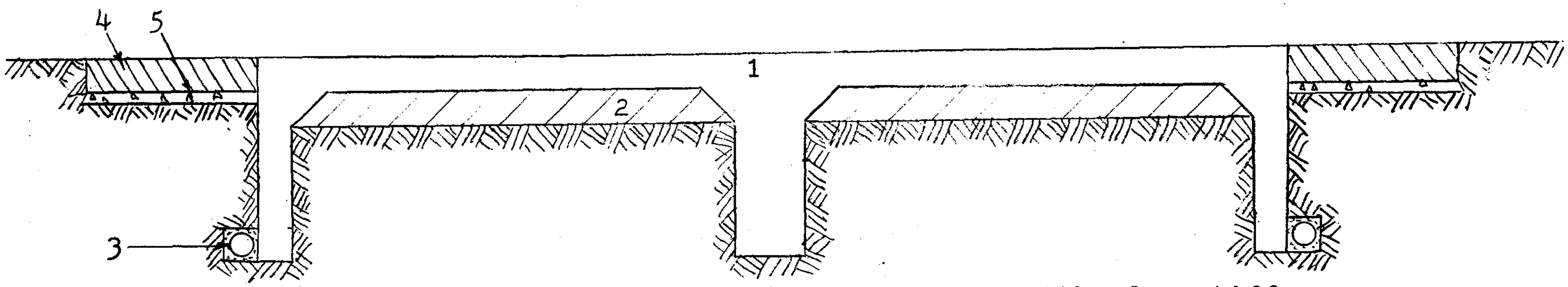
The first systems are suggested on Figures 83 and 84. The subgrade is prepared exactly in a manner similar to that suggested on Figure 82. The two proposed systems are slightly different. The first utilizes the variable thickness of the pavement slab concept while the other utilizes the edge stiffening and anchoring of the subgrade and subbase concept. We shall discuss the system shown on Figure 83 first.

Variable Thickness Pavement - The strengthened subgrade is overlain by an impervious membrane. Over this membrane is laid a porous subbase followed by an econocrete base and a steel fiber reinforced concrete whose top 3 in. (76.2 mm)

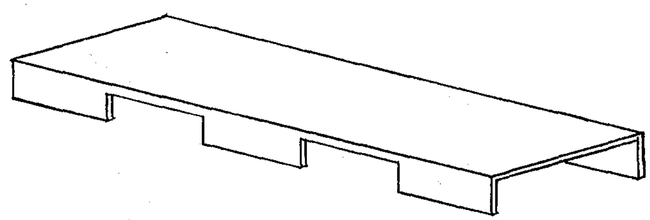


1 Steel fiber reinforced concrete, surface impregnated with polymeric substance
2 Econocrete base
3 Porous subbase
4 Impervious membrane
5 Drain pipe (approx. every 100' connect to lower pipe with vertical pipe)
6 Shoulder
7 Drain for subgrade (to lower water table)
8 Thermal insulation

FIGURE 83 VARIABLE THICKNESS PAVEMENT



1 Surface with edge stiffeners
2 Prepared base
3 Drain
4 Shoulder
5 Shoulder base



Note: Either Provide continuous edge strips or interrupted as shown.

FIGURE 84 ANCHORED PAVEMENT

76.2 mm or so are sulphur impregnated. The idea is to utilize the composite concept having the top layer of fibrous steel concrete with the lower layer of lower strength econocrete. If the construction does not utilize the composite concept, it is suggested that a polyethylene sheet or asphalt-rubber membrane be inserted between the econocrete and fibrous steel concrete so that the reflection cracks are arrested. Shoulders are provided, and a drainage system in the subgrade and subbase has been proposed. The drains in the subbase and subgrade can be connected to channel the water from the subbase into the subgrade pipes and then disposed.

This section is superior to the previous one in that edges of the pavement slab are stiffened with increased thickness. The additional thickness provides a higher resistance to flexure of the edges and corners that experience curling. The T-beam shape provides higher rigidity and promotes relative economy by reducing the thickness. The relatively economical econocrete can be used as a base.

It may be noted that, depending on the availability of materials, the econocrete base and porous subbase (shown as 2 and 3 on Figure 83) can be made of one material. Minor modification in the drainage system will then be required.

A modification of this system can be made by using "waffle-patterned" slabs instead of T-beams. The choice of these modifications will be based on topography, availability of materials, labor and other economic considerations.

Anchored Pavement - The second system is suggested on Figure 84. A prepared and strengthened subgrade is necessary over which a prepared base (econocrete, OGBAM crushed stone, etc.) is placed. A surface course with edge beams is constructed. The top layer of surface course is composed of steel fibrous concrete with the top 3 in. (76.2 mm) or so sulfur impregnated. Shoulders with shoulder bases are provided.

The edge strips can be continuous or can be interrupted, as shown in the lower part of Figure 84, though the confining effect in the subgrade will then be reduced. The provision of the edge beams will help to:

1. Minimize curling at edges;

2. Reduce stresses in slab by increasing stiffness;

3. Reduce the stresses in subgrade in general, and increase at isolated locations that will require special attention;

4. Confine and anchor the base and subgrade thereby increasing the stability of the whole system.

5.3.2.2 Systems Not Dependent on Subgrade Preparation

It may be pointed out that this system can be strengthened further by using a "waffle-pattern" slab instead of a plane slab. The waffle-pattern slab with the edge stiffener as provided will provide a relatively rigid slab. Such a rigid slab will require a very careful preparation of subbase and subgrade and selection of materials used in these, in view of the heavy stress concentrations in certain zones. (A rigid slab constructed over very weak subgrades is bound to cause distress in the slab.)

The second series of proposed systems is shown on Figures 85 and 86. All stresses from a pavement system are transferred to the natural subgrade, and the evaluation of strength and load carrying capacity of the subgrade are essential. It is felt, however, that the following systems will require relatively less work on the subgrade than the ones already described.

Waffle Base Pavement - Figure 85 is a reproduction of Figure 54, which depicts the "Anchorete" system developed by Walker. Our examination reveals that the system with minor modification has a great deal of merit. The system involves gridding the subgrade with 4- to 6-in. (10.16 to 15.24 cm) deep trenches that are on 7-foot (2.13 m) centers. Reinforcement is placed and econocrete can be poured, creating a continuous base material. Keys are formed in this base material to hold the surface course. A top layer of steel fibrous concrete can then be placed that will key into the base.

As the layer hardens, it contracts. Since it is keyed to the base, however, the contraction is inhibited, resulting in the numerous micro-cracks. The top layer can then be covered with polymer portland cement concrete (thin) layer or impregnated with sulfur. In this system, the base is relatively insulated from large thermal ingredients and, as a result, does not expand and contract as much. This reduces the number of cracks in the base. The cracks in the top fibrous concrete layer will be sealed by proper treatment procedures (PPCC or SIC).

Geometrical Pavement - Figure 86 proposes another unique concept. It utilizes a V-shaped trench to support pavement foundation. The V-shaped trench will have a porous layer (shown as 4 on Figure 86) encapsulated in a filter fabric over which will be laid a V-section -- either precast or cast-in-place (shown as 3 on Figure 86). This V-section can be made of econocrete, or any economical form of concrete, depending on the stresses derived from analyses. The space between the V-section and subgrade will have a layer of porous drain (shown as 5 on Figure 86) underlain by a filter fabric to stop the migration of fines. This porous drain layer can be made of crushed stone or any impermeable material.

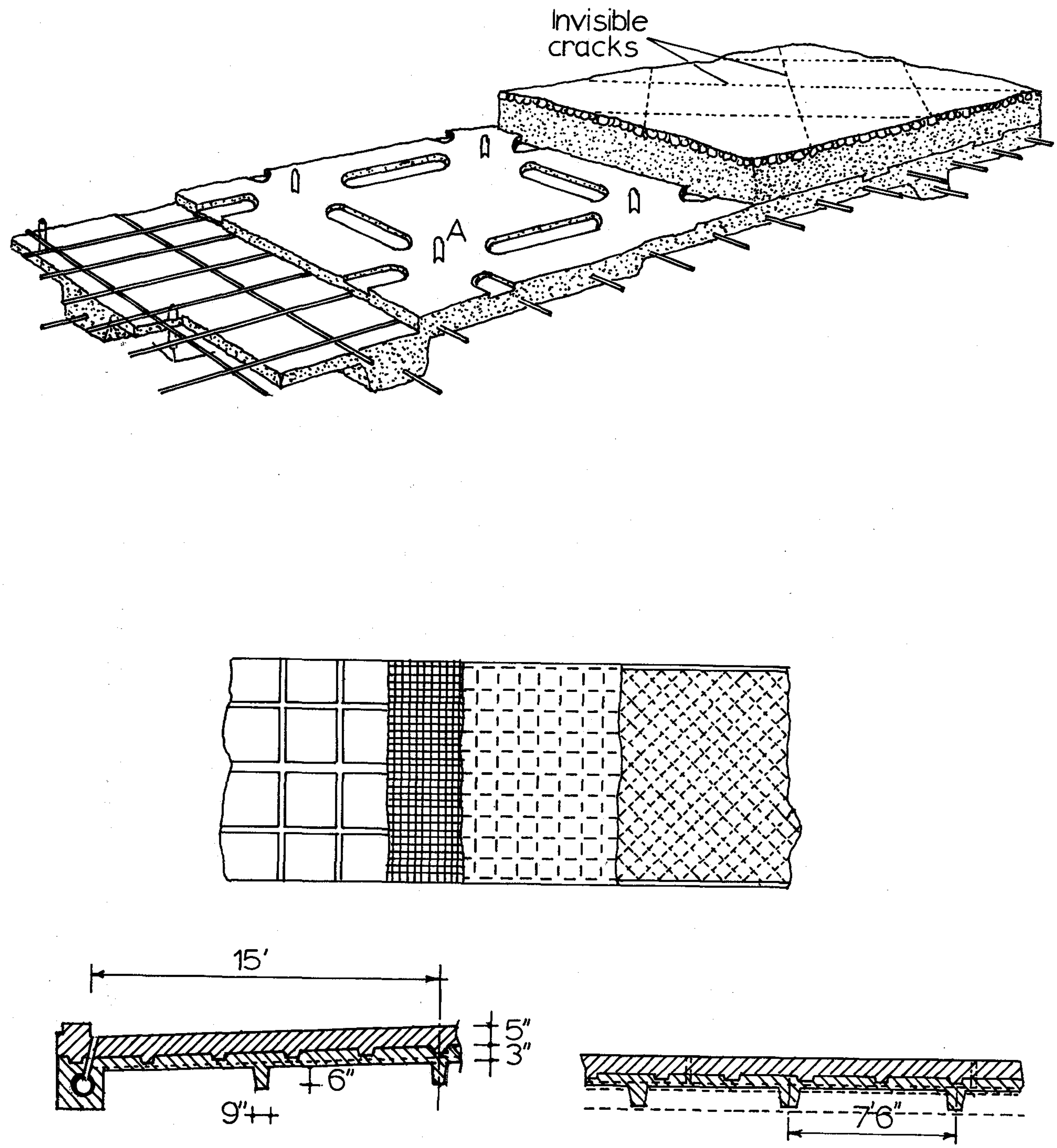
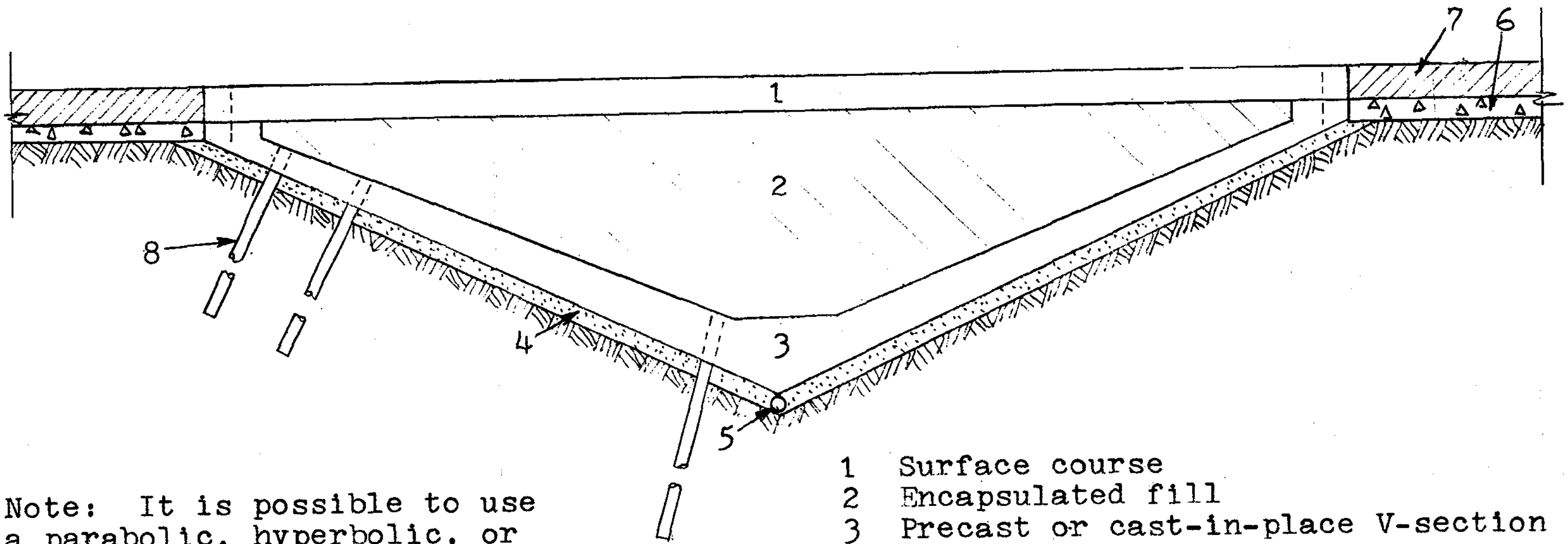


FIGURE 85 WAFFLE BASE PAVEMENT

(1 in = 25.4 mm)



Note: It is possible to use a parabolic, hyperbolic, or circular section instead of the triangular section shown.

1 Surface course
2 Encapsulated fill
3 Precast or cast-in-place V-section
4 Porous drain
5 Collector pipe
6 Shoulder base
7 Shoulder
8 Piles

FIGURE 86 GEOMETRICAL PAVEMENT

The top surface course is proposed to be fibrous steel concrete with sulphur impregnation. Shoulder surface and base layers are proposed.

To transfer the stresses from the V-shaped section, shallow friction piles are proposed (shown as 8 on Figure 86). The piles can be of wood, stabilized crushed stone or just by stabilizing the soil by different admixtures.

Even though a V-shape is proposed at this time, optimization of the shape as to parabolic, hyperbolic or any other form can be made after suitable analyses. Prefabrication can be used for the V-shaped section to ensure quality control as far as strength is concerned. The unevenness of different sections, as long as they can transfer stresses, is not of great consequence.

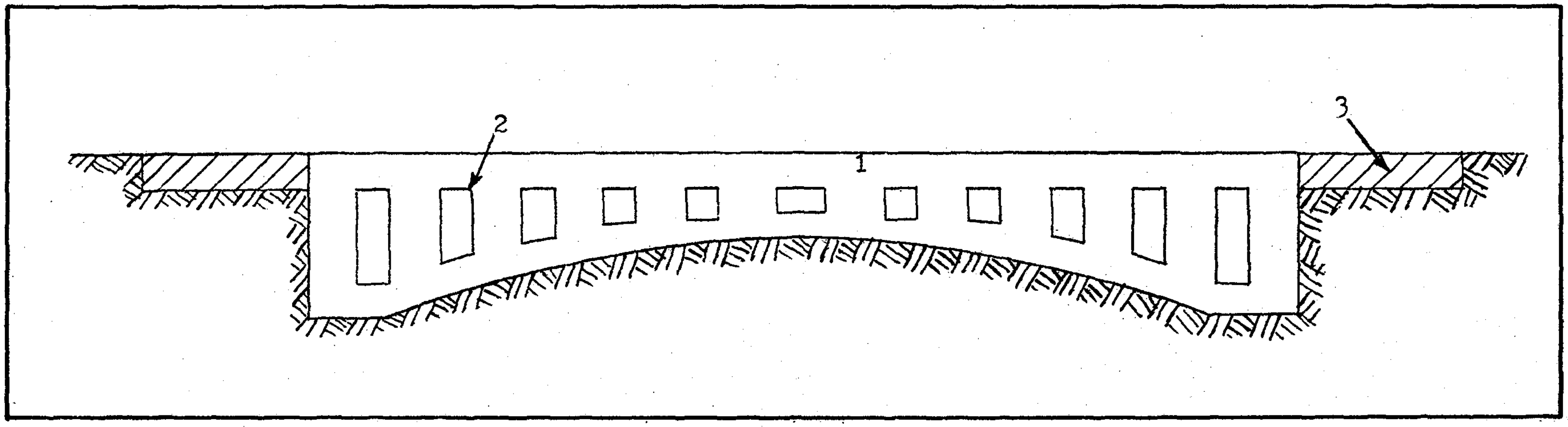
This system appears to have great potential. The system is universal and can be applied to any soil conditions. Of course,the thickness of the V-section, the number of piles and types of piles will be dependent still on the in-situ conditions and properties of natural subgrade.

Elaborate analytical work will be required to perfect the system, but we feel the system has great merit.

<u>Box - Section Pavement</u> - In Figure 87, another innovative concept is suggested. The system utilizes higher rigidity due to increased thickness yet is economic by utilization of rectangular, circular or elliplical hollow spaces. Such a structure is relatively rigid and therefore will tend to have higher contact stresses at the ends. Consequently, higher thickness has been assigned near the ends. The base shape proposed will tend to confine soil and reduce pumping. Such a section will require relatively little attention and detail in the preparation of the subgrade. The often encountered curling stresses will be minimal in such a shape, and the structure will maintain contact with the base or subgrade.

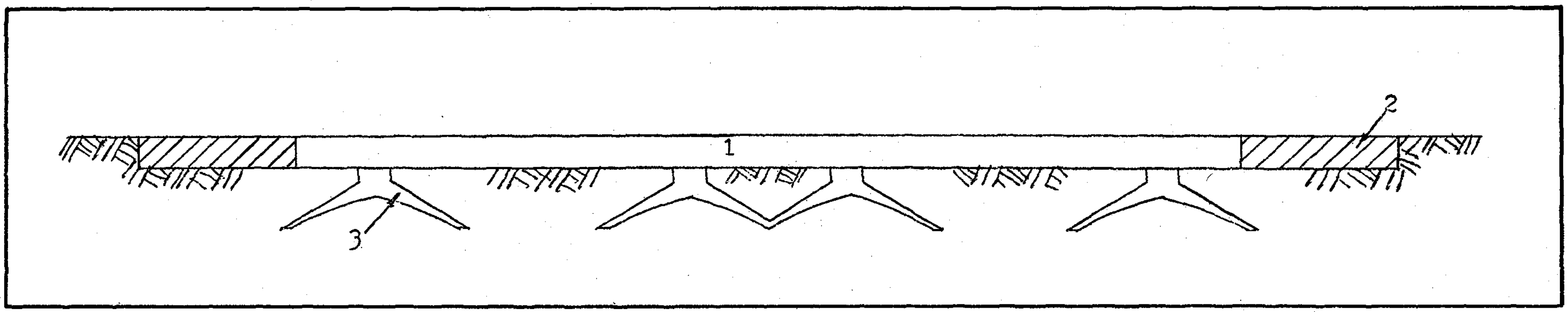
The top 2 to 3 in. (5 to 7.5 cm) can be made impermeable by any one of the several treatment procedures suggested already.

<u>Pavement on Hypar or Conical Strip Footings</u> - Figure 88 presents another novel idea. The idea was first suggested by Nicholls et al. (1968) for use in isolated footings. But the concept can be easily extended to strip footings. The membrane actions of hypar or conical shell considerably reduces the stresses transferred to the subgrade. The system provides ease of construction, confines the subgrade soil and can be used in almost any variety of subsoil. The concept requires further investigation in terms of optimization of shapes and actual dimensions.



1 Pavement structure
2 Rectangular, circular or elliptical patterns
3 Shoulder

FIGURE 87 BOX-SECTION TYPE PAVEMENT



1 Pavement
2 Shoulder
3 Hypar or conical footings

FIGURE 88 PAVEMENT ON HYPAR OR CONICAL STRIP FOOTINGS

5.3.3 Systems Where the Forcing Function is Modified

In the earthquake response to footings, it has been shown that if a footing is isolated by an elastic strip of rubber or other material, the intensity of earthquake waves to the footing is diminished. The elastic isolation modifies the forcing function. With this concept in mind, we have proposed a pile-supported pavement as shown on Figure 89. The structural slab (shown as 1 on Figure 89) to meet the zero-maintenance requirements is proposed of reinforced-fibrous-steel concrete with a sulfur or polymer impregnation. Two layers of energy absorbing elastic bearing (like elastomer bearings used in bridges) are placed between the pavement base and the pile cap to allow deflections and thereby damp out any vibrations by the transient wheel loads (shown as 3 on Figure 89).

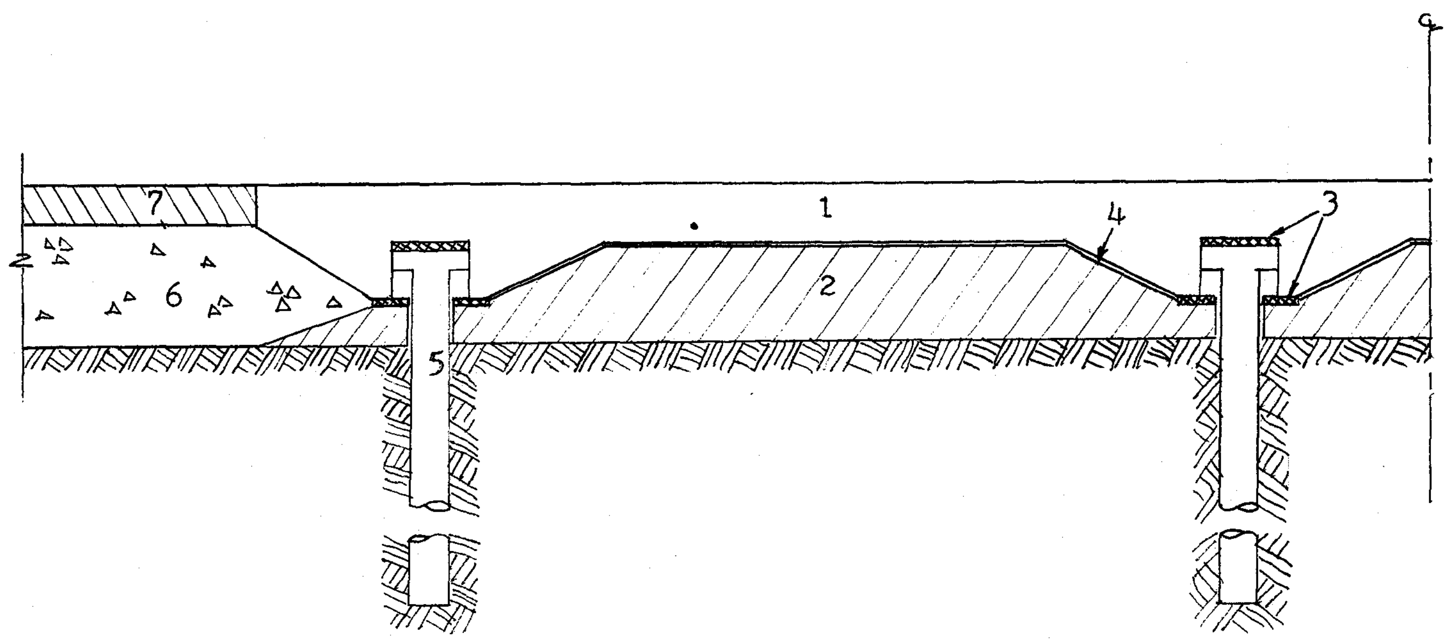
Plain or laminated neoprene bearings of many plane dimensions and thicknesses (consequently different hardness) are available. They can accommodate deflections and rotations in the vertical plane. Neoprene bearings are highly resistant to deterioration by weathering and natural aging and as is shown are simple to position.

The structural slab can be separated by a membrane or styrofoam (shown as 4 on Figure 89) to insulate it with the subbase. A base layer (shown as 2 on Figure 89) has been proposed to isolate the structural slab from the natural subgrade. The base layer could be of econocrete, OGBAM, etc., and where soil conditions warrant, can be designed to transfer a partial load of the surface slab to the in-situ soil.

Piles transfer the load to the subgrade by skin friction and/or bearing, depending on the soil condition. Piles can be of wood, steel, clean sand, weak cement-aggregate (the latter two can even be lime or cement stabilized). The intent is to transfer the major portion of the load by these pile-like inclusions to the deeper, hard bearing strata. The pile inclusion considerably reduces the horizontal tensile stresses in PC pavements. Surface deflections will be controlled dependent on the elastic bearings, which will in turn minimize the stresses in piles and consequently the subgrade.

5.3.4 Conclusions

Seven innovative and one modified pavement system (Figure 90) in conceptual form are proposed to meet the requirements of zero-maintenance pavements. Several modifications within these systems can be made. Some systems require strengthening and preserving the properties of subgrade during the life span of the pavement structure. Other are universally applicable without giving any serious



1 Pavement
2 Econocrete base
3 Elastic pads
4 Membrane
5 Pile
6 Shoulder base
7 Shoulder

FIGURE 89 PROPOSED PILE-SUPPORTED PAVEMENT

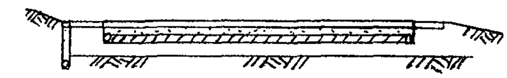
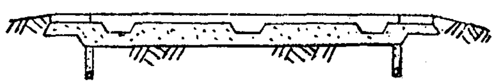
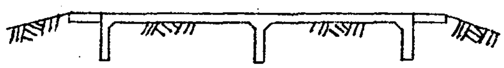
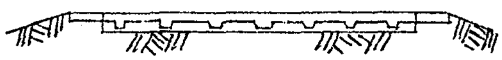
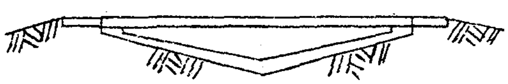
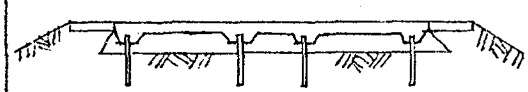
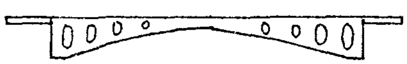
CONCEPT	PICTORIAL REPRESENTATION	DESCRIPTION
A		MODIFIED CONVENTIONAL WITH INSULATION AND DRAINS
B		EDGE-STIFFENED
C		ANCHORED
D		"ANCHORETE" GRID SYSTEM
E		V-SECTION PAVEMENT
F		PILE-SUPPORTED
G		BOX-TYPE PAVEMENT
H		PAVEMENT ON HYPAR OR CONICAL FOOTINGS

FIGURE 90 PROPOSED NOVEL SYSTEMS

treatment to the subgrade soils. All systems exhibit promise and potential, and can be utilized depending on the topography, labor conditions and availability of materials.

A series of analyses will be required to determine the actual stress distributions and optimize the dimensions and shapes of the constituents of the entire system.

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CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Many highways in and near urban areas are used by a huge volume of commuters; the demands of big cities in terms of commodities also requires heavy truck-traffic to come to the cities. As such, these pavements have to face heavy weights as well as large frequencies of heavy weights. This in consequence causes distress and premature failure. To repair a broken pavement (even a pothole) causes diverting traffic from one lane to another, which in turn causes traffic jams and very often some minor or major accidents. The need for pavements that can withstand large periods of maintenance-free performance under the unusual frequency and heavy weight of urban and commuter traffic cannot be disputed. While many parts of the so-called zero-maintenance pavement research is under progress by various agencies, the effect of this research is towards the identification of new structural systems for zero-maintenance pavements. To examine a structural system, it is necessary to examine the components of the structural system. The components of a pavement structure are: the subgrade, the subbase and base and the surface course. In this state-of-the-art report, a survey has been made of many new as well as infrequently used components of a pavement system. Based on the available data, their potential contribution to the zero-maintenance concept has been evaluated. The available information and the evaluation has been forged together to produce one modified conventional and seven new structural pavement systems that may be able to sustain urban traffic and be qualified for zero-maintenance criterion.

6.1 SUBGRADE

An ideal subgrade in a pavement system should be such that there are neither gross shear failures nor excessive deformations. The strength and deformation characteristics of the subgrade must be maintained during the life span of the zero-maintenance pavement. Natural subgrades are not of one type, and so different techniques are required for stabilizing the subgrades. This report has reviewed techniques that are relatively new and infrequently used. Established techniques for which state-of-the-art type reports exist have not been incorporated. However, depending on the type of soil, the technique that works -- may it be conventional or relatively new -- can be used. For improving the subgrade it has been suggested to utilize densification, strengthening, and protection against environmental factors.

6.1.1 Densification

The study points out as promising concepts: dynamic consolidation, vibroflotation and terra-probe. The two techniques -- compaction piles and compaction grouting -- need further study. The densification by blasting is perhaps not applicable in near urban areas.

6.1.2 Strengthening

The strengthening techniques of chemical injections and preconsolidation are and have been promising concepts. In addition, use of chemical admixtures such as cement, lime and bitumen, etc., have also been very successful. A method that requires further study is vibro-replacement (stone-columns). Methods that may have possible related applications are electro-osmosis and thermal treatments. Relatively little use has been made of these concepts although the concepts in themselves are sound.

6.1.3 Protection Against the Environment

Techniques and methods to encapsulate a subgrade by a membrane (membrane encapsulation) show excellent promise. Understanding of the need and improvement of drainage is accelerating and also the methods to provide suitable drainage are appearing. The insulation techniques have not yet been fully mastered. Styrofoam was used but in some places it gave bad results. More investigations into the area of insulation is warranted.

6.2 BASE AND SUBBASE

Cement, lime and lime-fly ash stabilized bases and subbases, though not discussed in this report, have excellent promise. Econocrete offers a promising concept worth developing in this contract. Porous layers of base and subbase (like OGBAM) exhibit promise in the concept of zero-maintenance pavement; however, a porous pavement surface (top course) is not applicable in the present context.

Fabrics used as a filteration media have been found to function well. Their strengthening characteristics have not been properly studied and therefore require further investigation. Some polypropylene fabrics have exhibited excellent control of reflective cracking phenomena.

Synthetic aggregates provide potential materials. Many of them, such as overburnt brick ballast, have been in use in many countries. The processed waste and other materials are potential concepts needing further study.

6.3 THE SURFACE

Steel fiber-reinforced concrete emerged as a promising concept for zero-maintenance pavements. Glass, cotton vegetable, asbestos and polymeric fibers were found not to be applicable in fiber-reinforced concrete to be utilized for the surface component of the pavement system under study,

Carbon fiberous concrete is a very worthwhile potential concept that needs further study. After study, this material may prove to be the best for zero-maintenance pavements.

Polymeric materials hold promise; however, it has been stressed in the recommendations to utilize the sulfur impregnation wherever possible in view of the availability of the material. Polymer impregnation, polymer portland concrete and polymer concrete-- all three will be used in some form in pavements most promising as an impermeable topping.

Waffle, griddled and composite pavements are potential concepts; however, they need more study. Similarly, the concepts of anchored pavements and additional thickness or reinforcement, though intuitively sound, need to be evaluated further.

Prefabricated systems have obvious advantages of better quality control. However, in view of the relatively large joints and other disadvantages, the investigators at the present time think it not to be an applicable concept as far as the surface layer is concerned. However, the prefabricated system may have a possible related application, for example in the subbase or base, in lower intensity traffic areas, and in detours where a quick construction may prove to be handy.

A study to investigate the suggestions or use of any innovative concepts in joints systems was made. Only few innovative suggestions were found. Nevertheless, the performance of even the known and prevalent joints is not accumulated and systematically evaluated. This is a task for the highway engineers.

Pavement on piles has been discussed in literature in very few places and so also has been rarely used. Nevertheless, intuitively thinking, such pavements cannot be excluded for consideration in a zero-maintenance system. More study is required.

6.4 RECOMMENDATIONS

While many side issues have been raised in this study that may in the long run be as beneficial as the main investigation, six new structural systems have emerged as a consequence of this study. One of the concepts is a modified conventional pavement structure and the other seven are relatively innovative concepts. These concepts are intuitively sound but need further analysis to be optimized.

It is recommended that a preliminary analyses of all these systems using linear theory and plane strain concept be performed on all systems. For comparitive purposes, the modified conventional system (Figure 82) should also be analyzed using asphalt concrete, portland cement concrete and fibrous steel concrete as the surface censor.

The preliminary analysis can then to utilized to evaluate the economy of a system and the two or three most economical systems be analyzed in detail.

APPENDIX I

FAILURE CRITERIA IN FIBER-REINFORCED CONCRETE

Griffith (1920) first derived the relationship for the so-called "brittle" materials between work of fracture, applied stress, and permissible crack length. His work has been generalized to include other materials. Fracture energy is conveniently measured in joules per square centimeter (J/cm^2) of crack area produced. Small but finite quantities of energy are involved in local tearing as a crack progresses through a material. This quantity is governed by the way the material in the vicinity of the crack is permanently distorted during the passage of the crack. Looked at in simple terms, it is a matter of force multiplied by the distance the material is moved or distorted. Griffith, when considering glass, was dealing with a solid having a fracture energy of the order 10^{-4} J/cm^2 and it seemed reasonable to equate this to the surface energy or surface tension of glass, which is of the same order. It was implicit that the work done was merely in separating two planes of atoms. More or less by definition, structural engineering materials to be stressed in tension have a work of fracture of 1-10 J/cm^2, a noticeable quantity of energy (1 J/cm^2 is about 5 ft $lb/in.^2$). A factor of 10^4 difference is beyond experimental error and implies distortion some distance in from the crack surfaces of a structural material.

Differences of this order make an exact calculation somewhat superfluous and so Griffith's treatment may be applied. The essential point is that a round crack with a solid, under a tension normal to the crack, has around it a roughly spherical stress-free zone, since no stress may be transmitted across the crack (Figure 91). For the crack to propagate, it must have a supply of fracture energy, which need not come from external work, since this can be arranged to be zero. Stored elastic energy is released as the stress-free zone is made larger by crack propagation, or put differently, the crack will propagate if the increment of elastic energy released is greater than, or equal to, the work of fracture needed. Now elastic energy stored per unit volume of material is half stress times elastic strain, or $\sigma^2/2E$, where σ is the average stress in the solid. As the crack changes length from $2L$ to $2(L + \delta L)$, (a) the volume of stress-free zone increases by $4\pi L^2 \cdot \delta L$, and (b) energy required is $2\gamma \cdot 2\pi L \delta L$ where γ is the work of fracture per unit area of crack face.

Since the volume term increases as a greater power of L than the surface term, we would expect a certain critical length for crack propagation when

$$4\pi L^2 \delta L \cdot \frac{\sigma^2}{2E} = 2\gamma \cdot 2\pi L \delta L$$

so that

$$\sigma^2_{crit} = 2\frac{\gamma E}{L} \quad \text{or} \quad L_{crit} = \frac{2\gamma E}{\sigma^2} \qquad \text{(Equation A-1)}$$

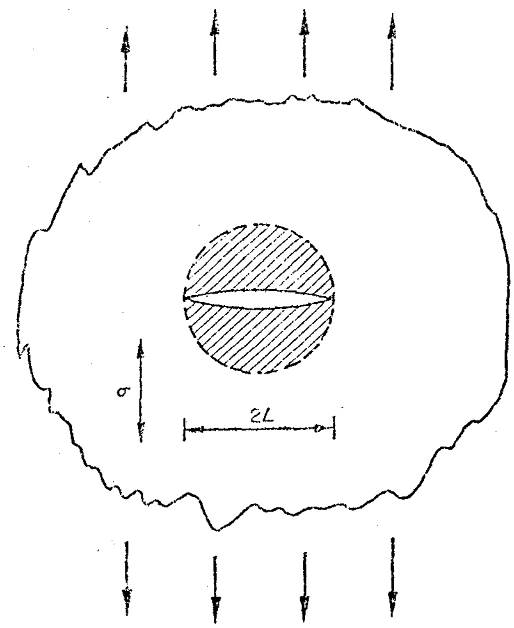

FIGURE 91   STRESS-FREE ZONE AROUND A CRACK (ISOTROPIC MATERIAL)

Now strengths of engineering solids generally lie in the range 0.1-1 percent of the modulus. Stresses can seldom exceed 1 percent E (excluding rubber) because of the need to limit deflection. Thus, we may write $L_{crit.} = 2\gamma/\sigma\varepsilon$ where ε is 10^{-2} to 10^{-3}. For an aluminium alloy with $\gamma = 1$ J/cm^2, say at a tensile stress of 70 MN/m^2 (10 ksi) and with E = 70 GN/m^2, the critical length is 30 cm, and such cracks should be readily detectable. For glass of the same modulus and at the same stress, the crack length would be about 0.035 mm. Raising the stress level to 700 MN/m^2 (100 ksi), which is possible for large pieces of pristine glass and just within the reach of modern alloys, brings the length down to about 3 mm for the alloy and 3000 Å for glass. Although one might have expected the result for glass, the business of locating cracks in a large structure with absolute certainty before they become 3 mm long is a daunting prospect, particularly as they might well start from any sharp corner or bolthole. Ideally, fracture energy should be markedly increased to match increases in working stress.

One way of avoiding this unstable crack hazard is to subdivide the structure or composite into a number of smaller component parts at which the crack is frustrated. However, it is not always possible to subdivide in this way, nor is it always feasible to raise the work of fracture as the strength of a solid is raised. If we assume that suitable composites are available, however, to what extent should the critical length calculation itself be modified? For a plane crack perpendicular to the stiff direction of an anisotropic material, the volume behind the crack, in which stress is zero or nearly so, will be greater (Figure 92). The spherical stress-free zone has become an ellipsoid, most probably with a major to minor axis ratio of $E^2/2G$, where E is the Young's modulus in the stiff direction and G the corresponding shear modulus in that direction. This ratio

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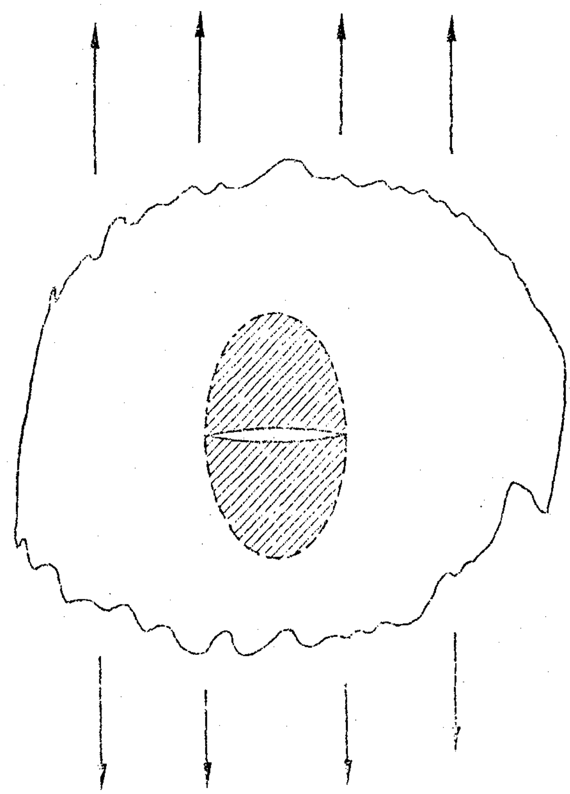

FIGURE 92 LOW SHEAR MODULUS INCREASES STRESS-FREE ZONE AROUND A CRACK (ANISOTROPIC MATERIAL)

direction of an anisotropic material, the volume behind the crack, in which stress is zero or nearly so, will be greater (Figure 92). The spherical stress-free zone has become an ellipsoid, most probably with a major to minor axis ratio of $E^{\frac{1}{2}}/2G$, where E is the Young's modulus in the stiff direction and G the corresponding shear modulus in that direction. This ratio term occurs in many similar problems of elastic stress transfer. The critical length for the crack to be unstable will then be reduced by the same factor relative to an equivalent isotropic term occurs in many similar problems of elastic stress transfer. The critical length for the crack to be unstable will then be reduced by the same factor relative to an equivalent isotropic solid. For fully aligned high-modulus fibers in resin, this length may be reduced to one seventh of that predicted by Equation A-1. The exact calculation has been made by Sih and Liebowitz (1968).

A similar conclusion may be reached by examining the increase in the concentration of stress at the tip of crack of given length, due to the anisotropy of the material. Savin (1961) deals with the general case of an elliptical hole in an elastic, anisotropic sheet. For example, he calculates that the peak tensile stress concentration, due to an elliptical hole of eccentricity 3, is 7.0 for isotropic media but as large as 18.7 when the tension is applied along the grain for spruce. This is because spruce has a ratio of longitudinal to transverse moduli of 20:1. (It is common to regard a crack in a sheet as an ellipse of large eccentricity.) Such a statement about stress concentration is more limited, however, and could only be interpreted for composites when the region of peak stress is large compared with an individual fiber and the composite entirely elastic in behavior.

Now reinforced resins are relatively weak in unreinforced directions, so that applied stresses must then be made small. However, the fracture energy of an unreinforced thermosetting resin is very small indeed (Table 24) and commonly observed. In practice, this effect is eliminated by cross-plying layers of fibers, thereby preventing stress relaxation behind the crack in the matrix phase. A mechanism by which increased work of fracture is achieved in composites a minimum condition for crack deflection is given exists in liteature but beyond the scope of our discussion.

A direct, practical assessment of the fracture toughness of a material is made by measuring the residual strength in the presence of a crack or notch of known length. The strength per unit length of crack may then by calculated, assuming only that residual strength will vary inversely with the square root of crack length. Thus, the fracture toughness is then reported in curious units, e.g., lb/in.$^2\sqrt{}$ in. (1 lb. = 4.45 N, 1 in. = 25.4 mm).

TABLE 24

FRACTURE ENERGIES OF SOME SOLIDS

Material	Fracture energy, J/cm^2
Steel	30 - 50
Aluminum alloys	1 - 5
Vulcanized rubber	1
Polymethyl metharylate	0.06
Resins (epoxy, polyester)	0.01
Graphite	0.005
Alumina	0.003
Firebrick	0.003
Glass	0.0005

APPENDIX II

RESPONSE OF FIBER CONCRETE SYSTEMS[1]

1.1 INTRODUCTION

1.1.1 Problem

The improved performance of fiber concrete systems relative to plain concrete is related to the post-cracking behavior of the fiber concrete. The tensile forces transmitted across cracked sections by the fibers maintain a continuity that is lost with cracking of plain concrete. However, the fibers that bridge the cracked section become exposed to an external environment, and fiber degradation may result. It is not possible to predict the effect of loss of fiber to the environment on the behavior of fiber concrete systems without a rational model of post-cracking fiber concrete behavior.

1.1.2 Objective

The objective of this study is to develop a model of post-cracking behavior of fiber concrete and to determine the qualitative effect of loss of fiber capacity to transfer tensile forces on the behavior of fiber concrete systems.

1.1.3 Scope

Two-dimensional models of fiber concrete structures are developed based on the available data on direct-tension behavior of fiber concrete. The response of a modulus of rupture beam and beams on elastic foundation is calculated for several hypothesized stress-elongation curves for fiber concrete with varying loss of fiber capacity.

2.1 MECHANICAL RESPONSE OF FIBER CONCRETE

2.1.1 General

The material behavior of fiber concrete has been deduced from the behavior of fiber concrete test specimens, which have usually been modulus of rupture beams. Load deflection response of fiber concrete flexural specimens generally has two stages, a linear behavior to a "proportional limit", which is followed by a significantly nonlinear behavior (ACI, 1973). A simple stress analysis based on the initial geometry of the specimen has been used to evaluate the first cracking stress and the ultimate flexural strength from the modulus of rupture beam data. This

[1]The work reported herein was undertaken by R. Abdel-Malck and James Lott of the Illinois Institute of Technology.

analysis is not applicable to post-cracking behavior of fiber concrete beams because the stress redistribution associated with the geometrical changes of crack growth and fiber bonding has been neglected.

A stress analysis of direct-tension specimens provides a better estimate of the material behavior of fiber concrete because the stress is approximately uniformly distributed across the specimen both before cracking and after fully developed cracking. The stress distribution is indeterminate during the random cracking process. Data from several studies (Shah and Rangan, 1971; Naaman et al., 1973, 1974, personal communication, 1974; Majumdar, 1974) of the response of direct-tension, fiber concrete specimens provide the basis for the model of fiber concrete behavior that is developed in this study.

2.1.2 Response of Direct-Tension Specimens

In the initial studies of the direct-tension, fiber concrete specimens, the movement of the test machine cross-head was measured and used as the fiber concrete elongation during testing (Shah and Rangan, 1971; Naaman et al., 1973, 1974). Typical load-elongation behavior for these studies is shown on Figure 93. This behavior is not representative of the material behavior of fiber concrete, since it corresponds to the resisting force developed in the loading grips and portions of the test machine as well as the fiber concrete specimen in response to an imposed cross-head displacement.

The drop in load at a constant elongation of Figure 93 is caused by the growth of a crack across the fiber concrete specimen. The constant elongation is the gross response of the test machine, grips, and fiber concrete specimen. The flexibility of the test system increases as the crack develops, and the force induced by the cross-head displacement decreases. The elastic recovery of portions of the test system in series with the cracking fiber concrete results in an approximation of fiber concrete behavior in a cracked region as shown on Figure 94. The actual stress-elongation behavior of the cracked region during the load drop cannot be determined because of the dynamic nature of the cracking process.

The elastic response of the system in series with the fiber concrete was physically removed from the measured response in a recent study at the University of Illinois at Chicago Circle Campus by measuring the fiber concrete elongation over a 101.6 mm. gauge (Naaman, personal communication, 1974). Idealized load-elongation behavior from this study is presented on Figure 95. The elongation over the 101.6 mm. guage continually increased throughout the test. The behaviors of Figures 94 and 95 are in qualitative agreement, but they do not represent the behavior of the cracked region of fiber concrete.

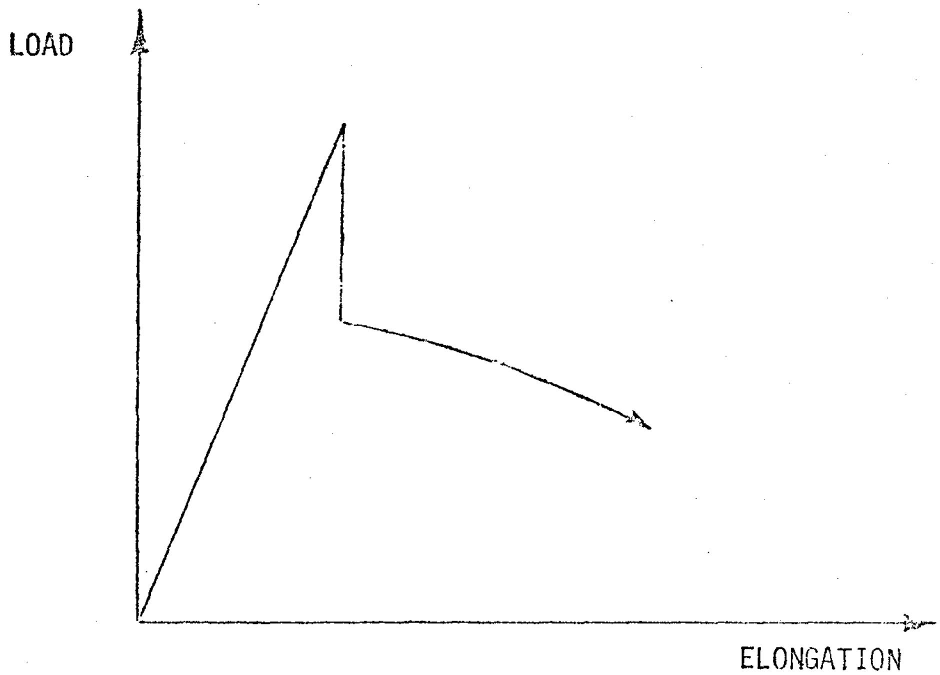

FIGURE 93
FIBER CONCRETE LOAD-ELONGATION CURVE FOR CROSS-HEAD MOVEMENT

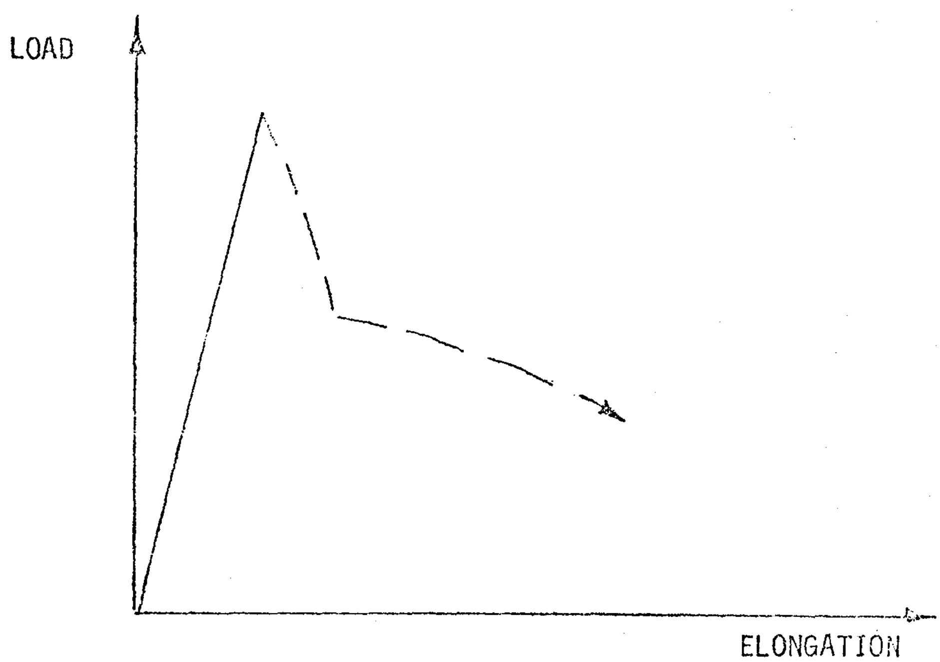

FIGURE 94
APPROXIMATE FIBER CONCRETE LOAD-ELONGATION CURVE

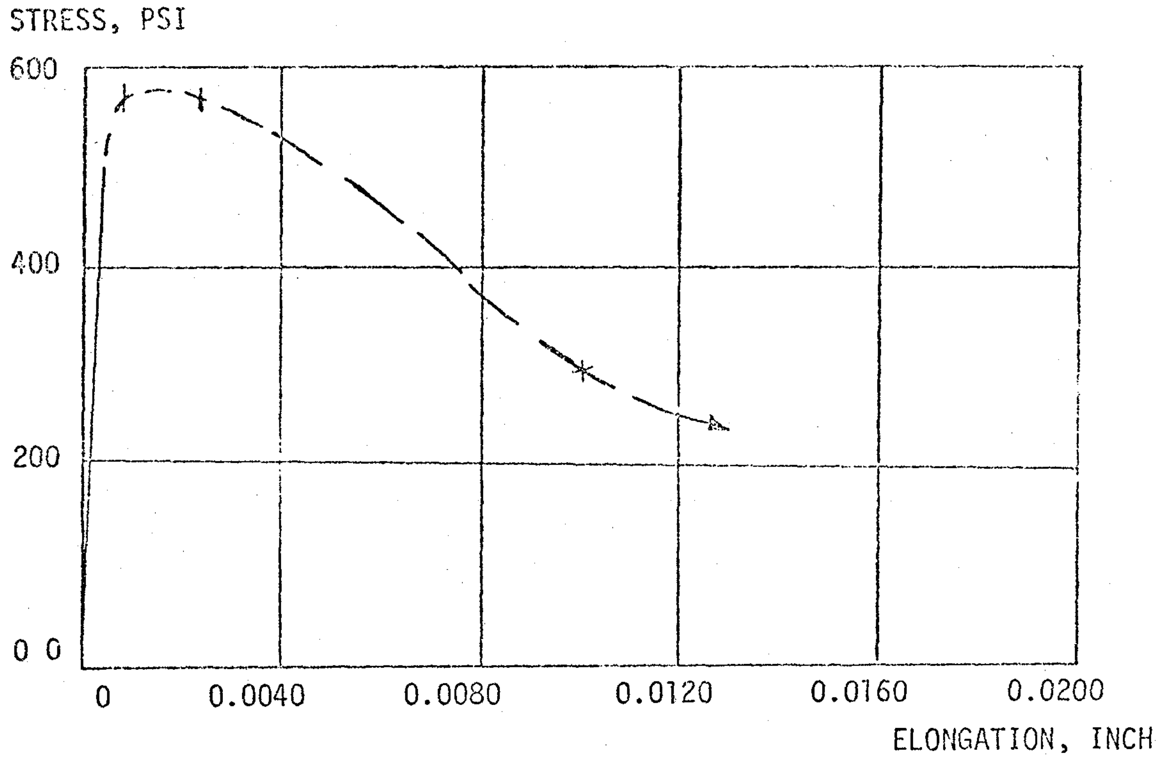

FIGURE 95

FIBER CONCRETE IDEALIZED STRESS-ELONGATION CURVE FOR FOUR INCH GAUGE
(1 psi = 6.9 kN/m^2; 1 in = 25.4 mm)

Load-crack width data have been obtained for a composite material with gypsum matrix and parallel glass fibers (Majumdar, 1974). Typical response data are shown on Figure 96 and are in qualitative agreement with the responses of Figures 94 and 95. The load-crack width data are the best available prediction of the behavior of the cracked region of fiber concrete. However, the data are for an approximate physical model of fiber concrete.

2.1.3 Tensile Behavior of Fiber Concrete

It is hypothesized that the fiber concrete in a prismatic specimen subjected to uniformly distributed axial tension progresses through four stages of behavior as shown on Figure 97. Stage I corresponds to macroscopic, linearly elastic behavior of the stess-elongation curve. On the microscopic level, fibers and concrete matrix deform together without cracking or unbonding. Stage II corresponds to the initial nonlinear behavior of the stress-elongation curve and is associated with the development of a crack and local unbonding at a critical, low-strength section. The crack development is a random process, and the stress redistribution during cracking is undeterminate. Stage III corresponds to the opening of the fully developed cracked section. The elongation in the region of the cracked section is the result of fiber elongation and slippage and is large relative to the elongations that occur in the noncracked regions. Stage IV represents the fractured specimen. The capacity to transmit tension across the cracked section is lost through a combination of unbonding or pull-out, fracture, and degradation of fibers at the cracked surface.

The response of fiber concrete as indicated by the stress-elongation curve is a function of the relative volumes of linear elastic material and of nonlinear, crack influenced volumes of linear elastic material and of nonlinear, crack influenced material that are present within the measured gauge length. Most of the material that undergoes geometric changes of cracking or unbonding is located within a distance of one-half of the fiber length of the cracked section because the fibers tend to pull-out on the side of short embedment. All analyses in this study are based on a 25.4 mm. gauge or element length, which corresponds to a commonly used length of steel fiber.

2.1.4 Idealized Stress-Elongation Curve

Two idealized stress elongation curves for a 1-in.(2.54cm) length of fiber concrete that contains a cracked section are presented on Figure 98 as Curves A and F. Curve A, which is used as the standard steel fiber concrete stress-elongation curve throughout this study, and Curve F are two of many possible interpretations of the 101.6 mm. gauge data of Figure 95 relative to the behavior of the fiber concrete within a 25.4 mm. gauge.

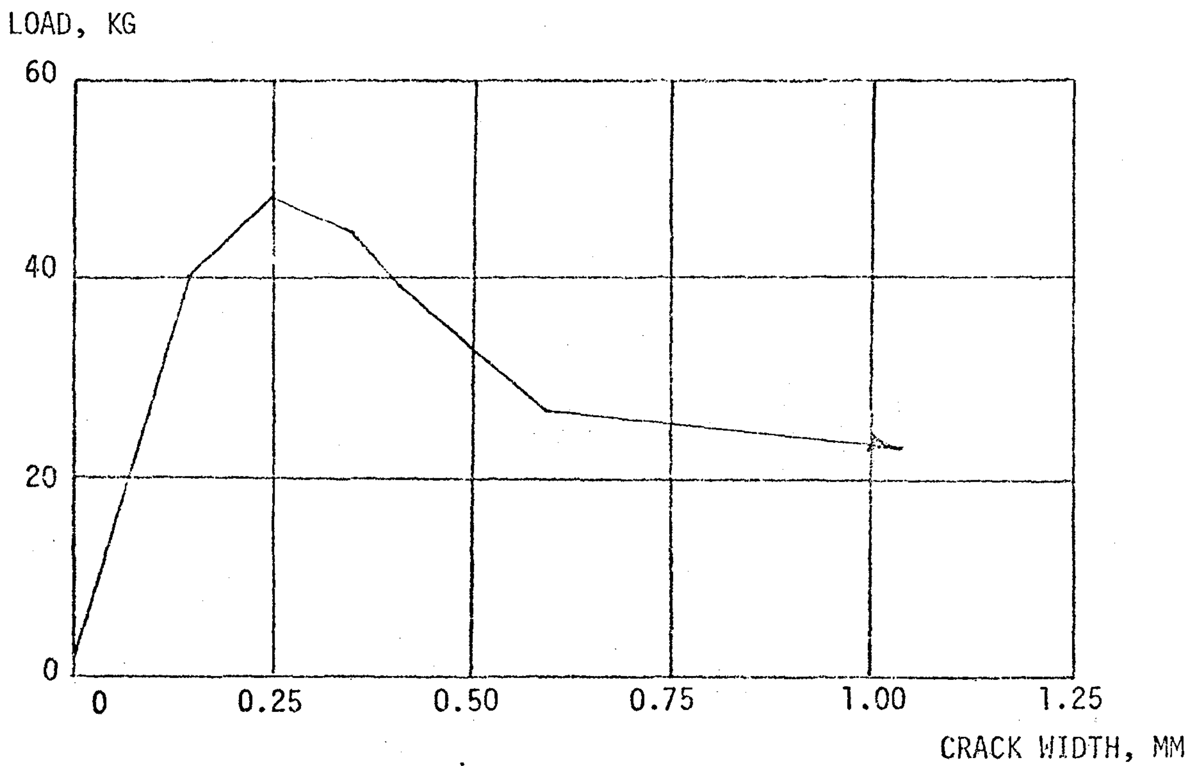

FIGURE 96

FIBER CONCRETE MODEL IDEALIZED LOAD-CRACK WIDTH CURVE

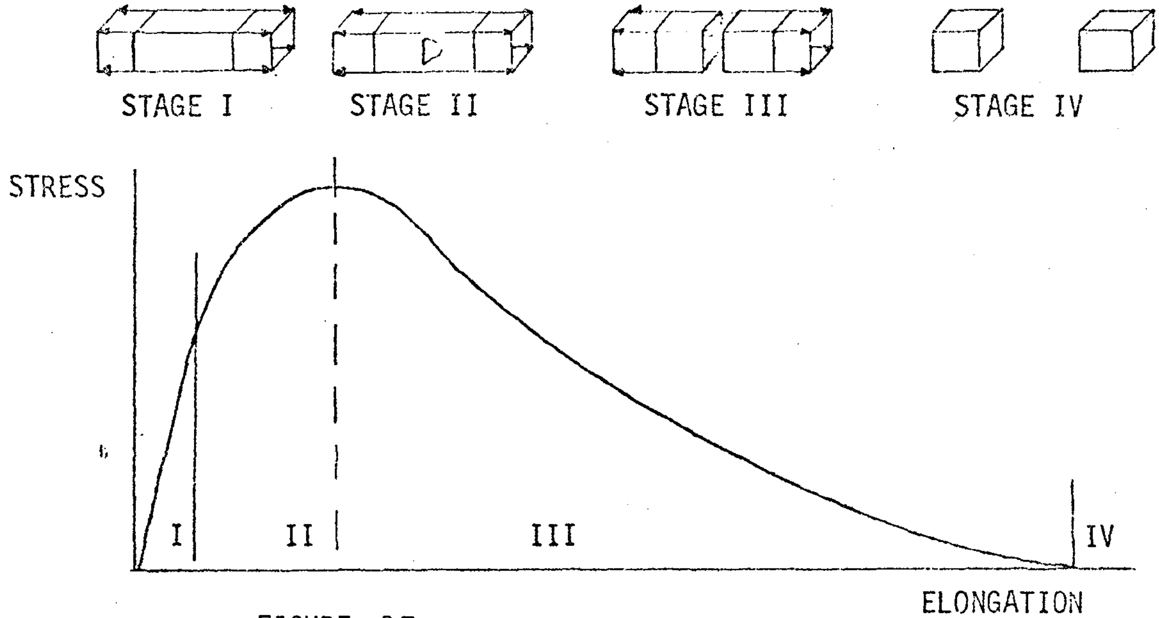

FIGURE 97
TENSILE BEHAVIOR OF FIBER CONCRETE

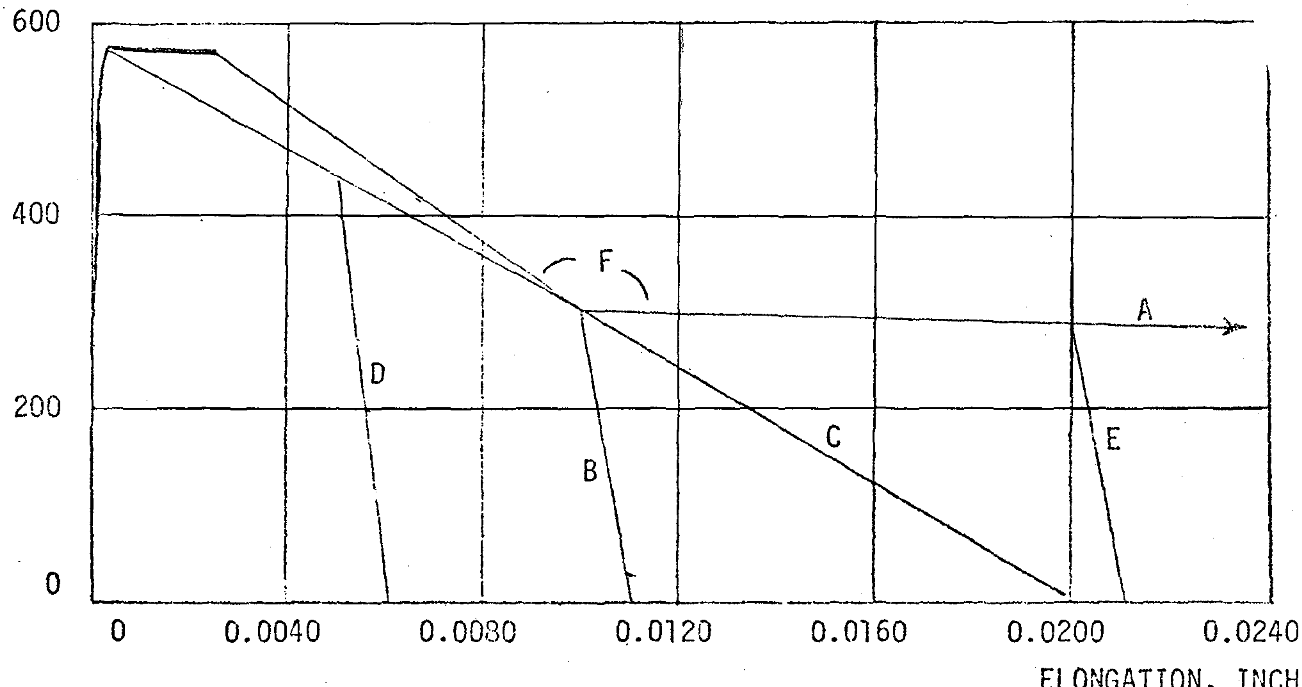

FIGURE 98
IDEALIZED FIBER CONCRETE STRESS-ELONGATION CURVES FOR ONE INCH GUAGE

(1 in = 25.4 mm)

The maximum stress of Figure 95 is 570 psi, and there is a relatively flat-topped region between elongations of approximately 0.0203 mm. and 0.0635 mm. over the 101.6 mm. gauge. The corresponding elongations within the 25.4 mm. gauge at 3933 kN/m^2 could range from 0.0051 mm., if the 0.0203 mm. elongation was uniformily distributed over 4 in., 0.0635 mm. if all of the 0.0635 mm. elongation occurred within the 25.4 mm. gauge. The lower limit of 0.0051 mm. elongation at 3933 kN/m^2 has been used in this study because it is conservative and predicts the least ductile material behavior.

2.1.5 Lost Steel Fiber Tension Capacity

Fibers that bridge a crack are exposed to an external environment that could degrade fibers with a resulting loss of fiber capacity to transfer tensile forces. The degradation is a complex function of environment, fiber composition, exposed surface area, and duration of exposure.

Four possible stress-elongation curves for lost fiber capacity are shown on Figure 98. Each curve represents a modification of the assumed standard behavior of steel fiber concrete, Stress-Elongation Curve A, which has been assumed to be a function of only elongation, which is an approximate measure of exposed surface area. Curve B has a linear loss of fiber capacity between elongations of 0.254 mm. and 0.279 mm.; Curve C has a linear loss of fiber capacity between elongations of 0.254 mm. and 0.508 mm.; Curve D has a linear fiber loss between elongations of 0.127 mm. and 0.1524 mm.; and Curve E has a linear fiber loss between elongations of 0.508 mm. and 0.5334 mm. These curves are proposed for stable conditions that are reached only after extended periods of exposure. The analyses that utilize these curves neglect the time-dependent nature of the stable condition, but they provide a conservative estimate of response.

3.1 MODELS OF FIBER CONCRETE STRUCTURES

3.1.1 General

Two models are developed for post-cracking behavior of fiber concrete structures. One model is for plane stress bending, and the second model is for a rectangular element loaded at the corners, which is used in a two-dimensional finite element analysis. Each model is based on the assumption that the hypothetical stress-elongation behavior of the fiber concrete prism subjected to uniform tensile stess corresponds to the behavior of fiber concrete in the more general case of a stress gradient. This assumption may not be valid because interaction or coupling among various regions is possible through common fibers. However, the assumption is reasonable for typical stress gradients in structures such as pavements.

3.1.2 Pure Bending

The model for plane stress bending is based on the additional assumption that strains or deformations in the fiber concrete beam are directly proportional to the distance from the neutral axis. The model is a 25.4 mm. length of beam that contains the cracked concrete section, and the relative displacements of the plane end sections are also directly proportional to the distance from the neutral axis. Since the strains and the displacements over the unit length are comparable, strains in the noncracked levels may be compared directly with the elongations occurring in the cracked levels.

Any given fiber concrete beam has a unique combination of resisting moment and rotation between the end sections for each value of strain or elongation at a given level of the beam. These unique combinations of moment and rotation result from stress and strain compatibility.

The moment-rotation combinations are determined for a given fiber concrete beam for several values of extreme tension fiber elongation. For a given extreme tension fiber elongation, the location of the neutral axis of the linear strain diagram is found such that the resultant force of the stress blocks is zero.

A typical moment-rotation curve is shown on Figure 99 for the standard steel fiber concrete stress-elongation curve. The shape of the moment-rotation curve resembles the shape of the fiber concrete stress-elongation curve. Each curve has continually increasing deformations with the load or stress increasing to a maximum value and then decreasing with further deformation.

The analysis of a fiber concrete structure incorporates a partial hinge with varying rotational stiffness at the location of a cracked section. The various moment-rotation combinations that occur at the partial hinge are given by the moment-rotation relationship of the 25.4 mm. beam length.

3.1.3 Modulus of Rupture Beam

The pure bending model is used to analyze a simply supported, fiber concrete beam that is loaded at mid-span. The load-deflection at mid-span curve is generated using various points on the moment-rotation curve. For a given combination of moment and rotation, the corresponding applied load is obtained from statics, the elastic mid-span deflection is calculated for this applied load, and the mid-span deflecttion associated with the rotation is also calculated. The total mid-span deflection is the sum of the two calculated deflections. The resulting load-deflection curve for a modulus of rupture beam has the general shape as shown on Figure 99 for moment rotation.

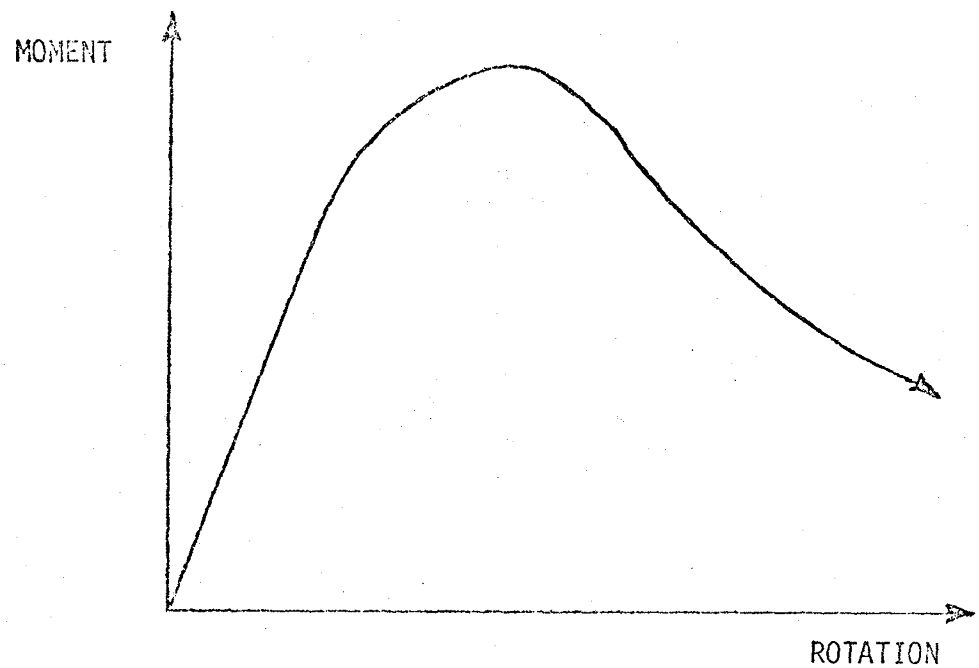

FIGURE 99

IDEALIZED MOMENT-ROTATION CURVE FOR ONE INCH LENGTH OF BEAM
(1 in = 25.4 mm)

3.1.4 Infinite Beam on Elastic Foundation

The partical hinge associated with the cracked fiber concrete section is incorporated in the analysis of beams on elastic foundation. The classic solution for an infinite beam on elastic foundation is modified by the partial hinge (Hetenyi, 1946). A load-deflection curve is obtained for a single concentrated load and deflection at the load using increasing values of rotation and corresponding moments from the moment-rotation curve for the 25.4 mm. beam element. For each combination of moment and rotation at the cracked section, there is a unique combination of applied load and deflected configuration that satisfies stress and strain compatibility.

Idealized load-deflection behavior for an infinite, fiber concrete beam on elastic foundation is shown on Figure 100. The large increase in deflection with no increase in load corresponds to the increased flexibility of the cracked section. Behavior of the cracked fiber concrete beam is bounded by the behavior of a fiber concrete beam with no hinge and with a complete hinge.

3.1.5 Rectangular Element

The model of the rectangular element which is loaded at the nodes (corners) is based on an assumption of linear variation in edge displacements between nodes. The element is considered to be cracked when the maximum principal stress at the centroid of the element exceeds the proportional limit stress of Figure 98. The cracked element is assumed to have the capacity to transmit tensile forces that are parallel to only one pair of sides of the rectangle. This implies that cracking develops in a plane normal to the pair of sides and that only normal stresses are transmitted across the cracked section by the fibers. Thus the loading of a cracked element becomes unidirectional.

A linear distribution of elongations normal to the cracked surface results from the assumed linear distribution of displacements between nodes. The elongations are a function of the nodal displacements normal to the cracked surface. Tensile stresses corresponding to the elongations are determined from the stress-elongation curve of the fiber concrete. The nodal forces that induced the elongations are obtained by statics. Thus, there is a unique relationship between the nodal displacements normal to the cracked surface and the nodal forces.

3.1.6 Finite Element Analysis

The finite element analysis of fiber concrete structures is conducted with a modification of the computer program VCSS (Volume Change Stresses in Concrete), which was developed at the University of Illinois (Naus et al., 1970). The original program considered a plane concrete element to be cracked when the maximum principal stress at the center of the element exceeds the

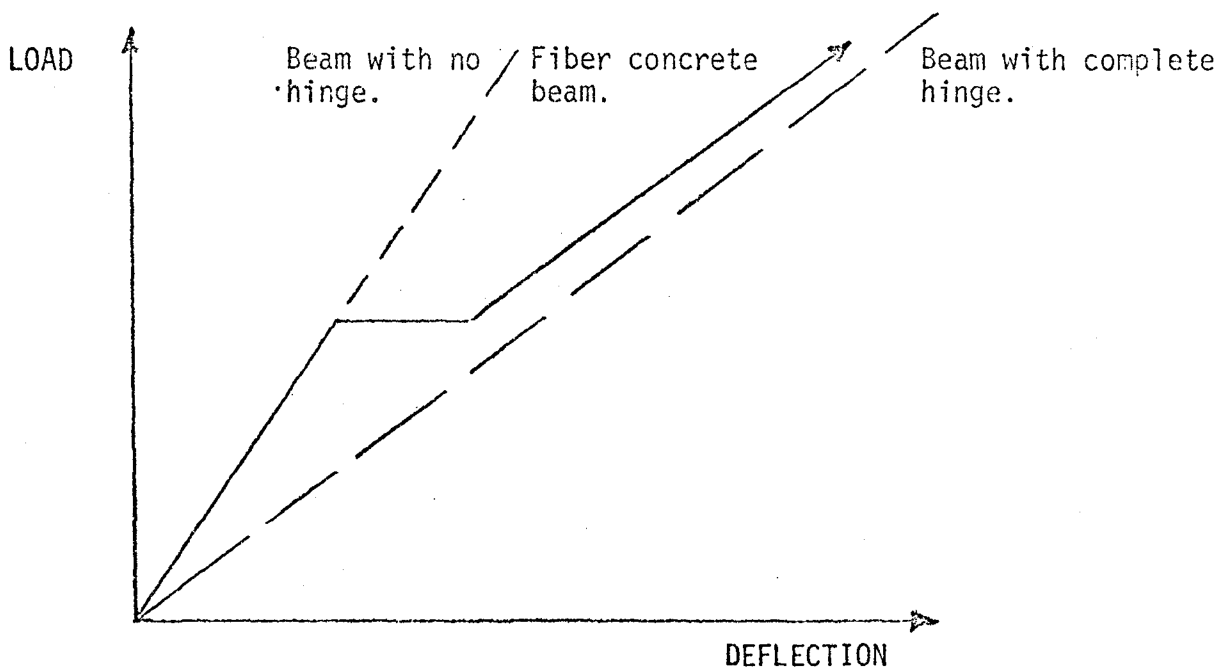

FIGURE 100

IDEALIZED LOAD-DEFLECTION CURVE FOR BEAMS ON ELASTIC FOUNDATION limiting tensile stress. The creacked element was removed from the system stiffness matrix, since it was assumed to have no load transfer capacity. In the modified program, the cracking of fiber concrete elements is limited to a single column of elements. When the maximum principal stress at the center of the element exceeds the proportional limit of the stress-elongation curve, an element is assumed to crack and is removed from the system stiffness matrix. However, nodal forces corresponding to the nodal displacements are applied to the adjacent elements since the cracked element still has tensile capacity normal to the cracked surface. Nodal forces are calculated from the displacements based on the assumption that the crack propagates upward through an element. An iterative procedure is used to analyze the cracked system with the addition of the new nodal forces, which are based on the displacements resulting from the previous loadings. The process is repeated until the average of the newly calculated nodal forces agrees with the average of the previously calculated nodal forces of all cracked elements within a prescribed limit.

The fiber concrete model of the modified program consists of equal sized rectangular elements of unit thickness. Boundary conditions are specified for nodal points, and the structure may be supported on an elastic foundation of constant stiffness. Loads are applied at nodal points. Steel reinforcement of given cross sectional area may be present along a single line of horizontal nodal points. The volume change aspect of the original program has not been utilized in the modified program.

4.1 PARAMETRIC STUDIES

4.1.1 General

The objective of the parametric studies was to determine the effect of a modification of the steel fiber concrete stress-elongation curve on the response to load of fiber concrete structural systems and to determine the stress redistribution that occurs within the fiber concrete structural system as cracking develops. The effect of the modified stress-elongation curves on response was studies for a modulus of rupture beam and for an infinite beam on elastic foundation using the pure bending model. Stress redistribution associated with cracking was studied for a finite beam on elastic foundation using the modified finite element program.

4.1.2 Modulus of Rupture Beams

The load-deflection behavior of a 152.4 mm. by 152.4 mm x 304.8 mm. fiber concrete modulus of rupture beam is shown on Figure 101 for the steel fiber concrete curves (A and F) and the four lost fiber capacity curves (B, C, D, and E) of Figure 98. The response of the beams for Curves A and F

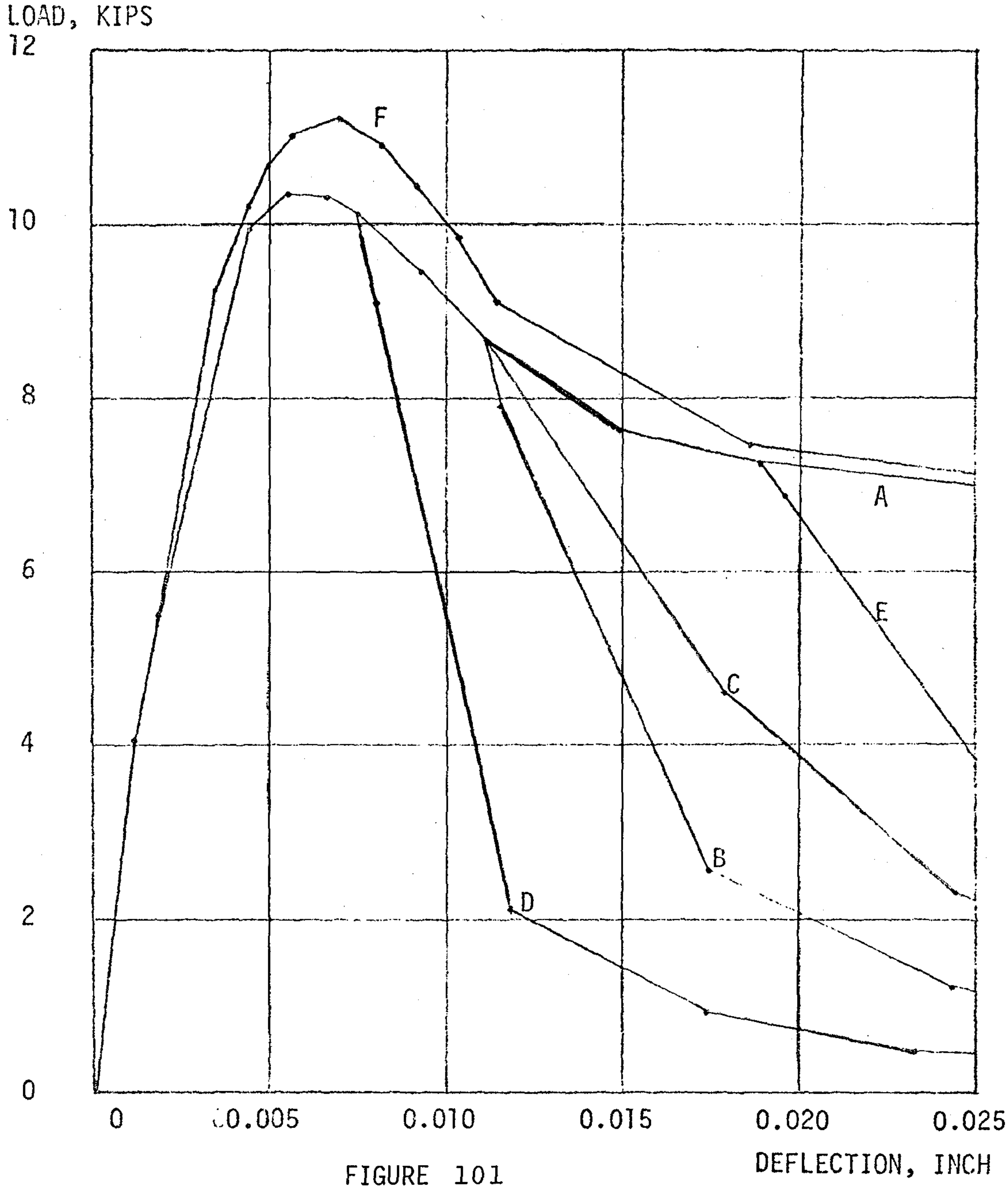

FIGURE 101

LOAD-DEFLECTION DATA FOR MODULUS OF RUPTURE BEAMS

(1 kip = 4450 kN/m^2; 1 in = 25.4 mm)

are similar, with Curve F having a higher maximum load and a greater elongation at maximum load. The response of beams for Curves A, B, C, D, and E are similar until the extreme tension fiber elongation reaches the threshold of loss of fiber capacity, when the load capacity of the beam decreases below the capacity of the beam with standard fiber behavior.

4.1.3 Infinite Beam on Elastic Foundation

An infinite beam of four inch (101.6 mm) depth and unit width on an elastic foundation of 11072 gm/cm^3 stiffness was analyzed using the pure bending model. The load-deflection at the load curves are presented on Figure 102 for the standard stress-elongation curve (A) and the four modified stresselongation curves (B, C, D, and E) of Figure 98. The dashed lines represent beams with no hinge, complete hinge, and complete hinge with no shear capacity next to the load. The cracked, fiber concrete beam response is bounded by the response of the beams with no hinge and with complete hinge. When the extreme fiber elongation reaches the threshold of loss of fiber capacity, there is a sudden increase in deflection toward the deflection of the beam with complete hinge.

4.1.4 Finite Beam on Elastic Foundation

One half of a fiber concrete beam of 101.6 mm. depth, unit width, and 64-in. length on an elastic foundation of 11072 gm/cm^3 stiffness subjected to two concentrated loads, P/2 located at 1 in. on each side of the center line was modeled with the column of cracked elements adjacent to the center line. The analysis was conducted for the standard stress-elongation curve (A) and for one of the modified stress-elongation curves (B) of Figure 98.

The two load-deflection curves are presented on Figure 103. Dashed lines again correspond to beams with no hinge, complete hinge, and complete hinge and zero shear capacity next to the load. The load-deflection behavior of the finite beams is similar to the behavior of the infinite beams as shown on Figure 102.

Deflection configurations of the cracked fiber concrete beam with the standard stress-elongation curve, of the beam with no hinge, and of the beam with complete hinge are shown on Figure 104 for increasing applied loads. As the load increases, the partial hinge develops in the fiber concrete, and the deflection configuration of the fiber concrete beam shifts from that of the beam with no hinge toward that of the beam with complete hinge.

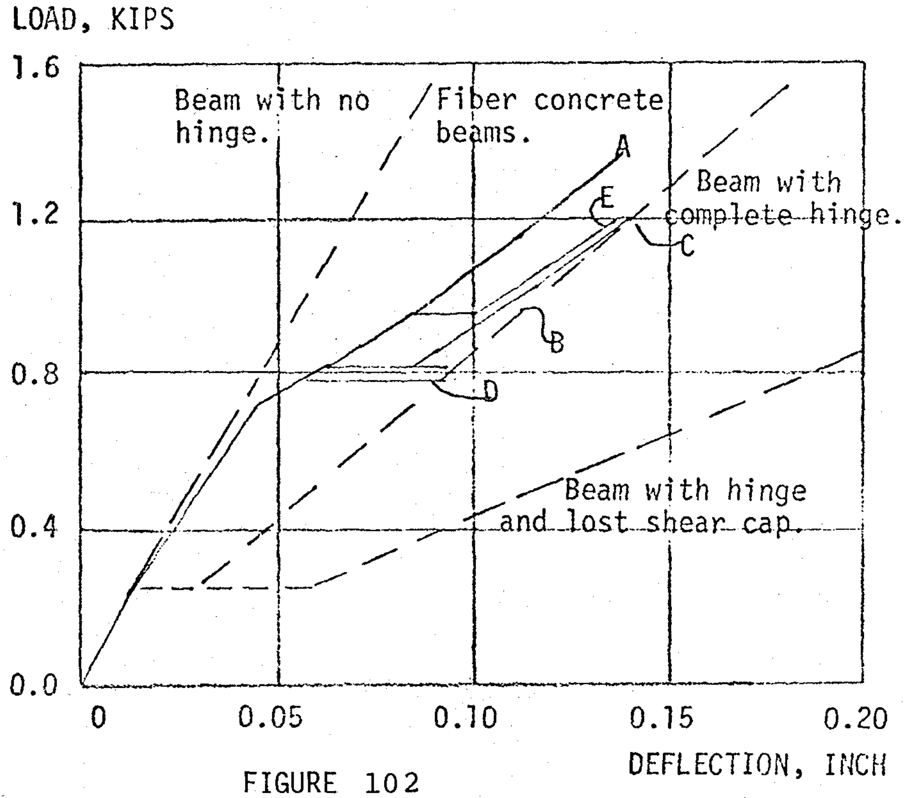

FIGURE 102

LOAD-DEFLECTION DATA FOR INFINITE BEAMS ON ELASTIC FOUNDATION

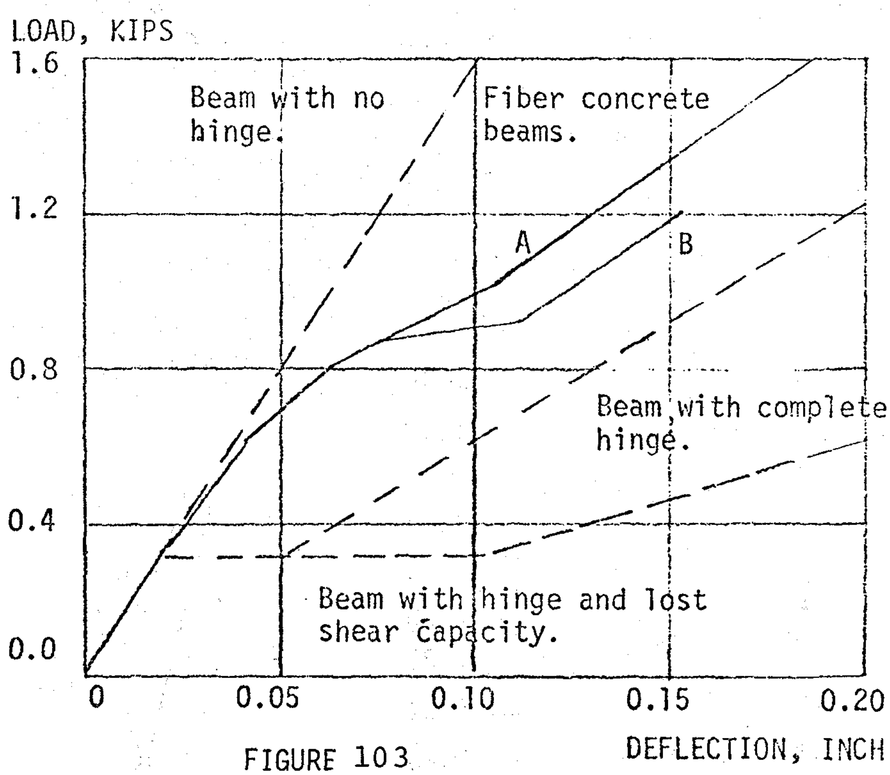

FIGURE 103

LOAD-DEFLECTION DATA FOR FINITE BEAMS ON ELASTIC FOUNDATION

(1 kip = 4450 kN/m^2; 1 in = 25.4 mm)

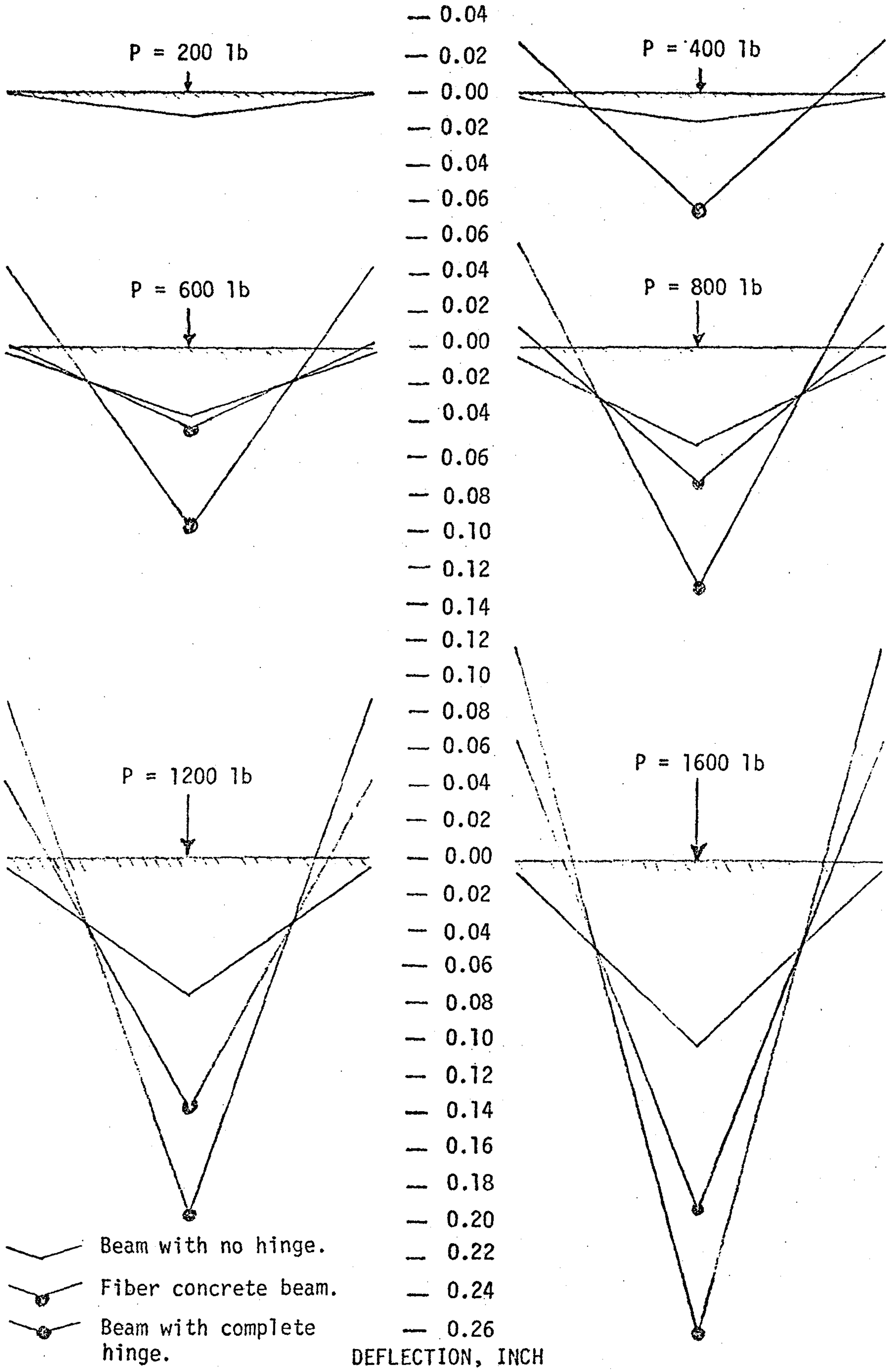

FIGURE 104 DEFLECTION CONFIGURATIONS OF FINITE BEAMS ON ELASTIC FOUNDATION
(1 lb = 4.45 N; 1 in = 25.4 mm)

The above deflection configurations are repeated on Figure 105 for the load level of 5340 newtons. Deflection configurations for the fiber concrete beam with lost fiber capacity and for the beam with complete hinge and lost shear capacity are also shown on Figure 105. Loss of fiber capacity causes a shift from the deflection configuration of the beam with standard stress-elongation curve toward the configuration of the beam with complete hinge.

Stresses normal to the cracked surface are shown on Figures 106 to 109 for the centroid of elements in the region of cracking. Maximum crack width (elongation) at the extreme tension fiber and the nodal forces transfered through the cracked elements are also shown on Figures 106 to 109. The beam with the standard stress-elongation curve is shown on Figures 106 to 109 for the load levels on the half beam of 445, 890, 1335, and 1780 newtons, respectively. The tensile stresses in elements adjacent to the cracked elements are influenced by the fibers. Loss of fiber transfer capacity results in lower tensile stresses adjacent to the cracked elements.

4.1.5 Discussion

The analysis of fiber concrete structural systems with various idealized fiber concrete stress-elongation curves provides insight to the response and to the stress redistribution of fiber concrete structures in the post-cracking region.

Loss of fiber transfer capacity results in the loss of the remaining load capacity of fiber concrete modulus of rupture beams as shown on Figure 101. This lost load capacity occurs on the descending portion of the load-deflection curve of the beam with standard stress-elongation curve. Thus the loss of fiber capacity results in a loss of beam toughness rather than a loss in beam strength.

The loss of fiber capacity in beams on elastic foundation results in a decreased stiffness at the cracked section and corresponding increases in deflection at the load as shown on Figures 102 and 103 and rotational increases as shown on Figure 105. The increased deflections cause increased soil stresses near the load, but soil stresses remain lower than the soil stresses for the beam with complete hinge.

The stress redistribution that occurs with cracking in the finite beam on eleastic foundation is most complex because of the interaction between the soil stiffness and the varying beam stiffness. Tensile stresses in regions adjacent to the cracked elements are related to the tensile forces transmitted across the cracked section by the fibers.

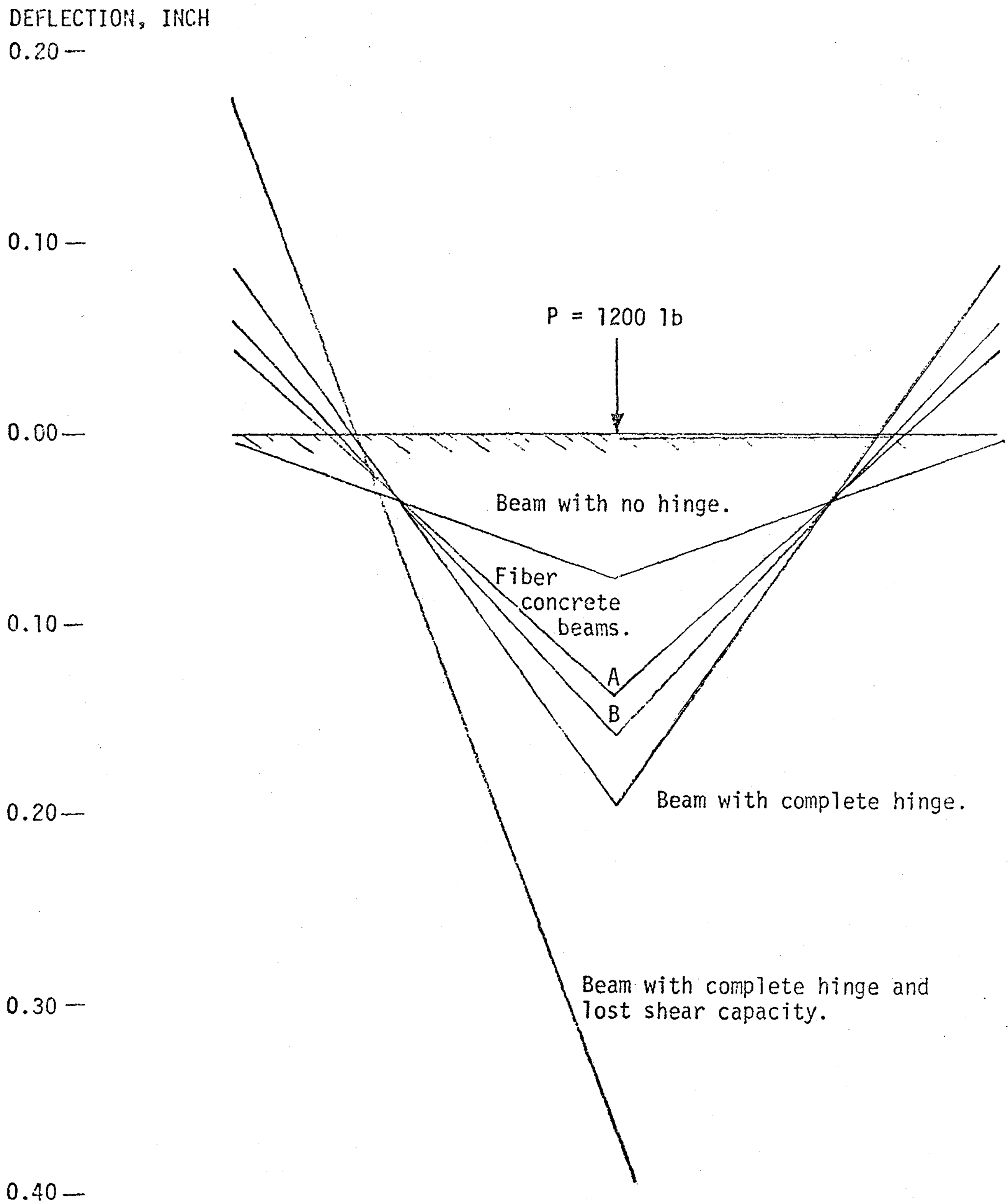

FIGURE 105

DEFLECTION CONFIGURATIONS OF FINITE BEAM ON ELASTIC FOUNDATION

(1 lb = 4.45 N)

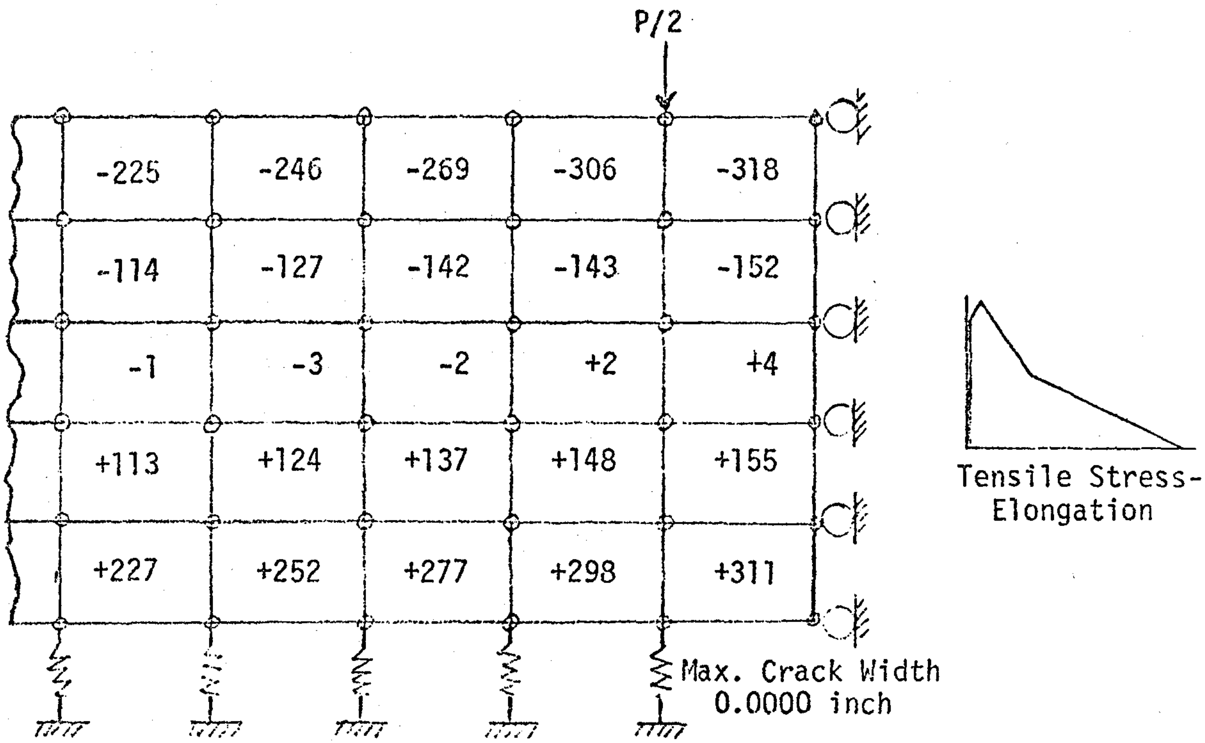

FIGURE 106
NORMAL STRESSES FOR P/2 = 100 POUNDS- STANDARD STEEL FIBER CURVE A

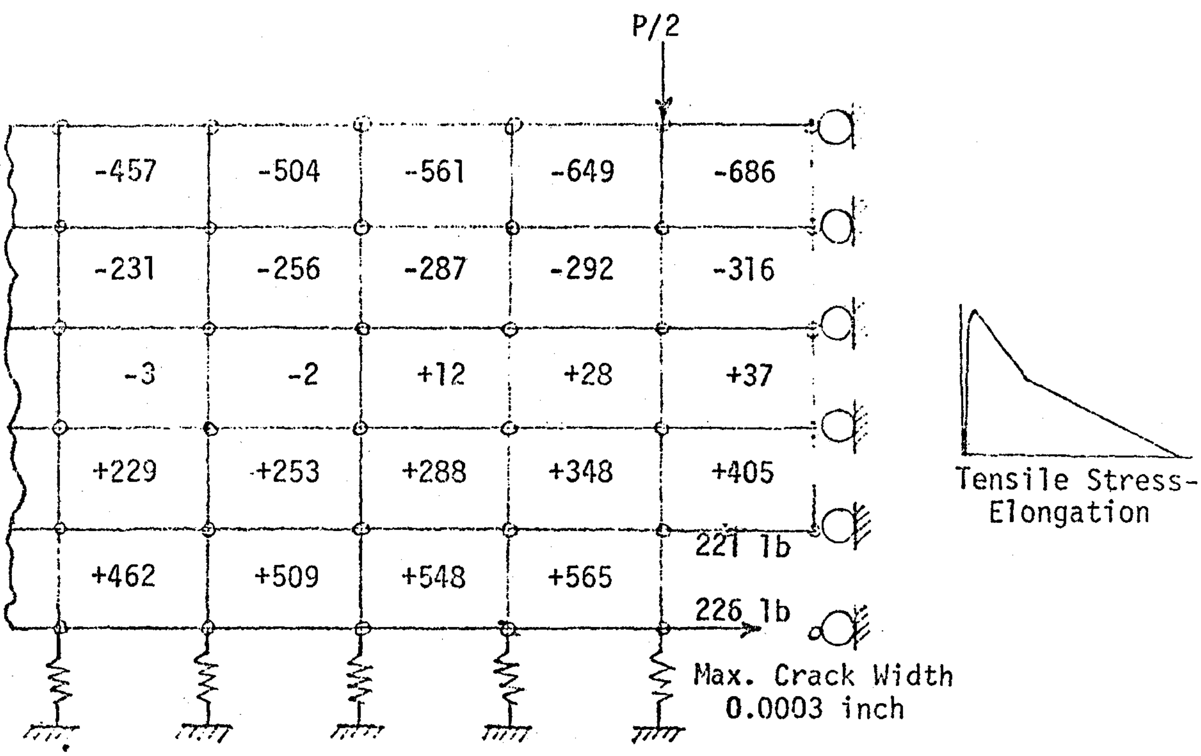

FIGURE 107
NORMAL STRESSES FOR P/2 = 200 POUNDS- STANDARD STEEL FIBER CURVE A

(1 lb = 4.45 N; 1 in = 25.4 mm)

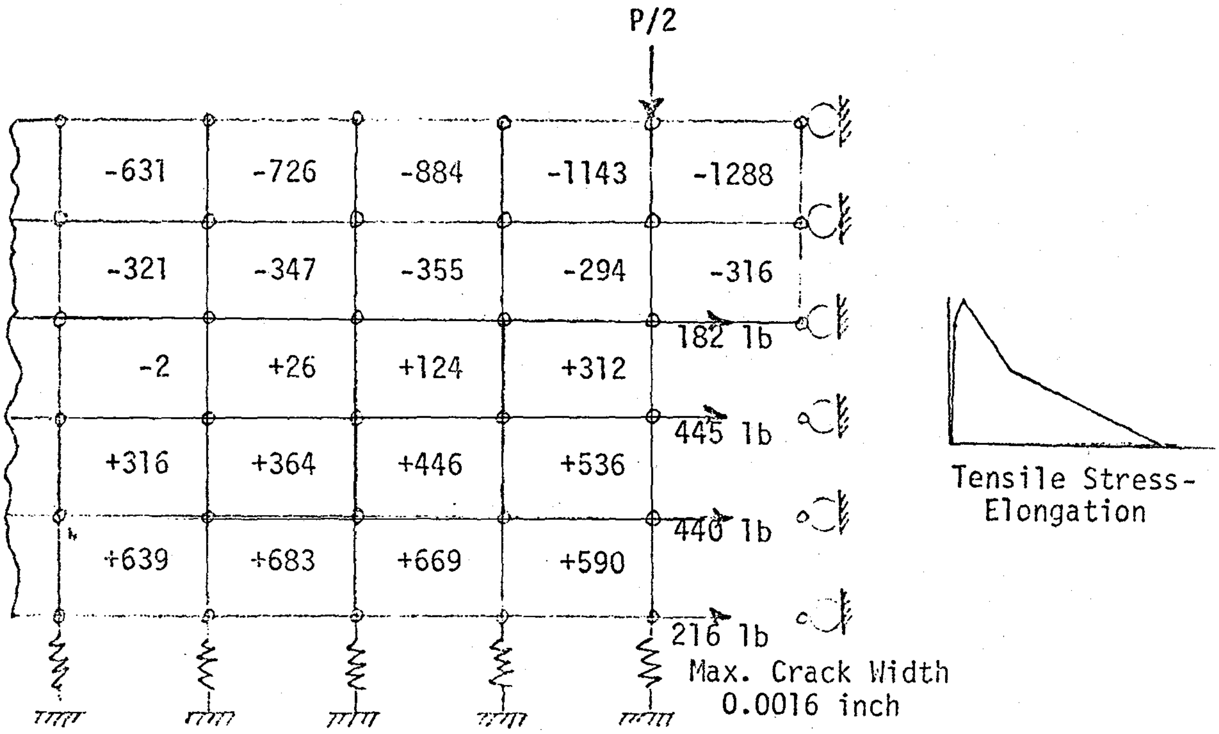

FIGURE 108
NORMAL STRESSES FOR P/2 = 300 POUNDS- STANDARD STEEL FIBER CURVE A

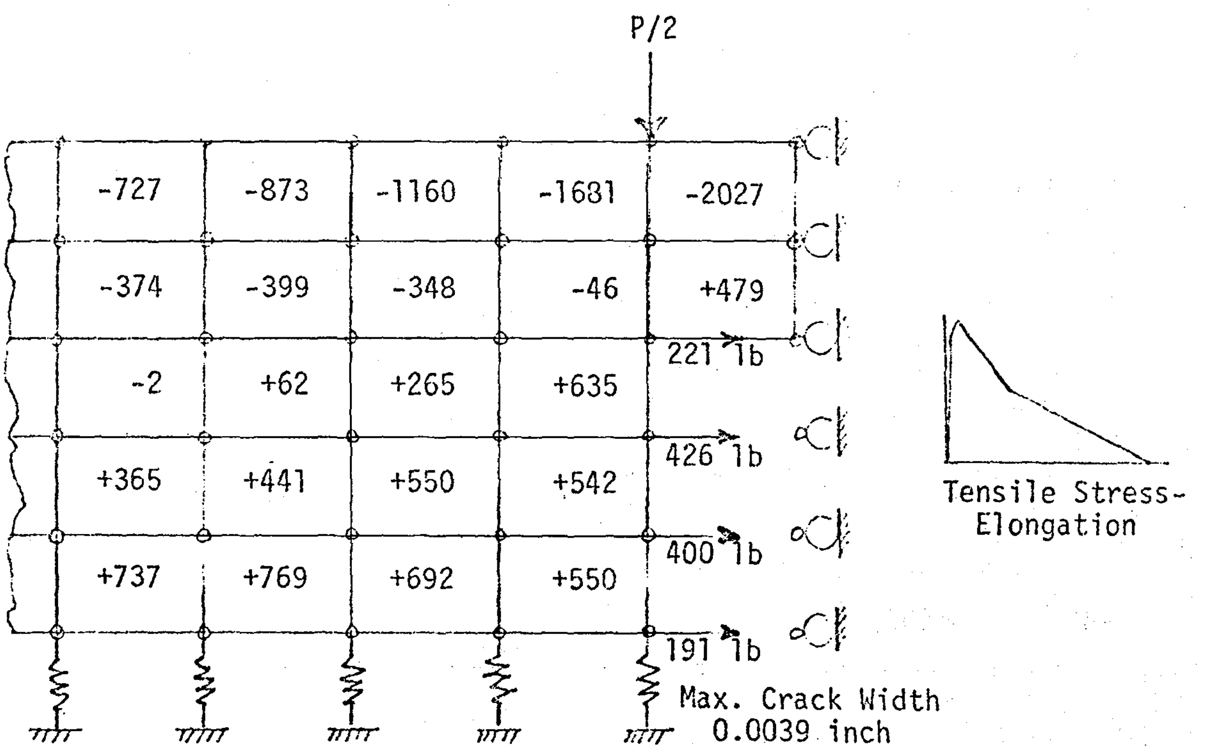

FIGURE 109
NORMAL STRESSES FOR P/2 = 400 POUNDS- STANDARD STEEL FIBER CURVE A

(1 lb - 4.45 N; 1 in = 25.4 mm)

5.1 CONCLUSIONS

It is possible to qualitatively predict the response of cracked fiber concrete structures for various tensile transfer capacities of the fibers at the cracked section. For the assumed idealized stress-elongation curves of steel fibers with various loss of transfer capacity, the effects are as follows:

1. Loss of fiber capacity at the cracked section of a modulus of rupture beams results in loss of beam toughness but not beam strength;

2. Loss of fiber capacity at the cracked section of a beam on elastic foundation increases the soil stiffness relative to beam stiffness and tends to reduce fiber concrete stresses in the region of the crack. Beam deflections and soil stresses in the region of cracking increase;

3. Loss of fiber capacity has a greater effect on the modulus of rupture beam than on the beam on elastic foundation because increased flexibility of the beam on elastic foundation is compensated for by the increased soil stress.

6.1 APPLICATIONS AND NEEDED RESEARCH

6.1.1 General

The simple models that have been developed may be used to compare the post-cracking behavior of a fiber concrete structure to a similar structure of portland cement concrete. The quantitative comparison of simple models may be qualitatively extended to more complex, acutal structural systems to evaluate fiber concrete applications.

Application of the fiber concrete models is limited by current knowledge of the post-cracking behavior of fiber concretes in various environments.

6.1.2 Application to Slab on Grade

Parametric studies of the beam on elastic foundation, which would be similar to the studies of Section 4, will indicate combinations of design parameter that provide fiber concrete beams with equal or improved performance relative to portland cement concrete beams of various depths.

The performance of fiber concrete slabs on grade relative to plain and reinforced concrete slabs may be qualitatively predicated based on the findings of the parametric studies. Thus, the behavior of an important type of structural system to be constructed of fiber concrete may be evaluated relative to behavior of a concrete system, with which the engineering profession is familiar.

6.1.3 Application to Reinforced Concrete Beams

The finite element analysis of fiber concrete systems may be utilized to evaluate the cracking behavior of fiber concrete beams that incorporate standard tensile reinforcement. Crack width and length and the stiffness of the cracked beam may be studied for various combinations of reinforced concrete and fiber concrete parameters.

The analysis will only be valid for beam response before yielding of the tensile reinforcement. The important case of flexural cracking under service loads is within this limitation. The analysis is also limited in that unbonding of the tensile reinforcement occurs only within the length of the cracked element. Thus, various unbonded lengths for reinforcement may be obtained only by changing the length of the cracked element, which also affects the system response.

6.1.4 Research Needs

Refinement of the computer program is not required until a better understanding of the post-cracking behavior of fiber concrete is obtained. Research into the behavior of fiber concrete is needed in three major areas before rational design procedures can be developed for practical applications. Load transfer-deformation data across cracks in fiber concrete subjected to both tension and shear is required to obtain accurate evaluations of structural response. The loss of fiber capacity to transfer force through prolonged exposure of cracked sections to adverse environments is needed to predict the interaction of fiber concrete with deformed reinforcing bars to reduce splitting at the reinforcement should be investigated to determine the concrete reinforcing bars force transfer mechanism, which could improve structural reliability of reinforced concrete beams.

6.2 REFERENCES

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