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ITIFICATION OF CANDIDATE ZERO MAINTENANCE PAVING MATERIALS

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FOREWORD

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

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

Charles F. Scheffey

Director, Office of Research

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16. Abstract

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

The other volume of this study is:
Report FHWA-RD-77-110, Vol. 2.
It contains chapters 9-15, pages 264-634.

17. Key Words

zero maintenance, asphalts, concretes, special cements, sealants, ceramics, prestressed pavements, vacuum processing, polymers, fiber reinforcement

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PREFACE

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

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

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

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

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

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

IDENTIFICATION OF CANDIDATE ZERO MAINTENANCE PAVING MATERIALS

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Maintainability is a built-in characteristic of any physical system and can be defined as a measure of the effort needed to maintain the system. Pavement maintainability is usually measured by the annual maintenance costs for the life of the pavement. Often a maximum acceptable amount of maintenance is specified for a particular pavement.

In highway design, trade-offs exist between initial costs and future costs. The principal future costs for a highway are maintenance costs and road user costs. In the past, the options in design have usually been either high initial cost and low maintenance costs or lower initial costs and higher maintenance cost. Unfortunately, future maintenance costs are usually difficult to estimate. Even if the future maintenance effort could be determined, it's satisfactory implementation may be constrained by budgets limited for administrative organizations that are incapable of performing the work, or by administrative or political decisions to make low maintenance a goal. To minimize these potential sources of difficulty, it would be ideal to begin with a pavement that requires little or "zero maintenance." Zero maintenance as used in this report refers only to the structural adequacy of the pavement system.

Zero maintenance or low maintenance is not a goal which should be sought to the point where its implementation requires a disproportionate increase in construction costs. Increased first cost could be justified, within limits, if it produced decreased rates of deterioration and, hence, decreased needs for maintenance and repair and later replacement. Estimates have shown, for example, that an additional $500K per mile would be justified on a high volume, six-lane urban expressway if the time to first overlay it for maintenance reasons could be doubled.

Some pavements made with conventional paving materials have been in service for 30 to 40 or more years with very little, if any, significant need for maintenance and repair. These could be looked upon as examples of wasteful overdesign or as examples of cases where locally available economical materials were of unusually high quality and the construction techniques were nearly perfectly adapted to the environmental conditions of use. The truth is probably somewhere between the two, however. These pavements generally have not been in heavy traffic regions. Increased pavement use is associated with increased rates of deterioration and increased need for maintenance and repair and earlier replacement. In some instances, pavements have required maintenance, repair, or replacement very soon after construction due to a single specific deficiency -- either one involving failure to follow requirements of specifications or failure of the specifications to require what was needed. The first case is something which must be handled individually for each job. The second case requires proper design for anticipated loads and environmental conditions. A material for zero maintenance pavements is a part of this proper design.

Many new materials or composite materials are developed each year for a wide variety of end uses and some of these could probably be adapted to pavement systems if costs were not a consideration. Regarding these materials, the underlying question raised by the United States Federal Highway Administration is:

> Can new materials be used to construct a pavement which will provide satisfactory "rideability" to exceptionally heavy traffic without requiring maintenance for "X" years, where "X" may vary from 20 to 40 years.

By "new materials" is meant materials other than those used in conventional portland cement and bituminous concrete pavements which can be used to form a materials system for structural pavements. The addition of a single new material to the conventional materials sytems qualifies the system as a new materials system. Both precast and field-formable materials systems can be used. Ideally, the new materials system should exhibit, in addition to the desirable qualities of present concretes, the following characteristics:

a. A rapidly developed significant tensile strength (rigid systems) or sufficient elasticity to accept vehicle and environmental stresses without cracking or distortion (flexible systems).

b. Low thermal expansion characteristics.

c. Low volume change upon placement.

d. Low volume change upon wetting and drying.

e. Greater durability in the pavement environment.

Above all, the new materials system should not result in significant increases in the initial cost of construction. Moderate increases, offset by anticipated benefits, are acceptable.

1.2 OBJECTIVE

In order to attempt to answer the question raised by FHWA, the following research objectives were established:

Task A. Conduct a detailed literature survey to determine the new materials which are available for adaptation and use in highway construction. This search shall cross technical lines to include materials which may not have been previously considered for highway application. Consultations with technical experts shall be held, if necessary, to procure information on unpublished work.

Task B. Expand Task A to include data of other researchers and examples of highway applications of these materials. Rate the materials on the basis of their properties, ease of construction, ease of maintenance when necessary, availability, costs, and environmental compatibility.

Task C. Prepare similar rating sheets of Task B [illegible] conventional portland cement and bituminous concretes.

Task D. Using the data and experience on new materials systems, recommend the research design for future detailed developmental and refinement work on these materials. This recommendation should include detailed listings of the testing which is necessary for incorporating these new materials into construction with reasonable assurance that they will not fail prematurely.

1.3 SCOPE

1.3.1 Task A. A number of meetings were held at the Waterways Experiment Station (WES) among the various staff members who had an expertise relating to the type of materials and materials systems which could be considered as candidate zero maintenance paving materials. The findings of these meetings resulted in a categorization of possible zero maintenance materials into six groups:

Category	Material or Materials System
1	Fiber Concrete, Polymer Concrete, and Expansive-Cement Concrete
2	Ceramic Materials
3	Sealant Systems
4	Noncalcareous Inorganic Cements
5	Special Construction Techniques
6	Organic Cementing Systems

A more detailed breakdown of each category is contained in Table 1.1. The listing shown in Table 1.1 represents the findings of Task A and is the plan for further consideration of candidate materials for zero maintenance.

As is the case in most research, other materials or materials systems became apparent as work progressed. These included such materials systems as nonasphalt organic binders for flexible pavements, super water reducers for concrete, rolled fill concretes, and others. Especially intriguing are combinations of the various materials already under consideration. With engineering imagination, the list will continue to grow with time and for the pruposes of this report it was only feasible from both a cost and time standpoint to consider only those items noted in Table 1.1.

1.3.2 Task B. The expansion of the categories of Table 1.1 into detailed information represents the main thrust of Task B. This information is found in the following sections of this report:

Category	Description	Chapter
1	Expansive Cement Concrete	8
	Fiber Reinforced Concrete	9
	Polymers in Concrete	10
2	Ceramic Materials	12
3	Sealant Materials	11
4	NonCalcareous Inorganic Cements	6
5	Prestressed Concrete Pavements	13
	Vacuum Processed Concrete	14
6	Gussasphalt	3
	Asbestos Asphalts	4
	Sulfur Modified Asphalts	5

Some of the items listed in each category may not be described in the appropriate chapter due to lack of information on the material or materials system, but all were given consideration and an attempt made to document the findings.

1.3.3 Task C. The characteristics of asphalt concretes and conventional portland cement concretes are briefly summarized and documented in Chapters 2 and 7, respectively. No attempt was made to document the innumerable applications of these materials.

1.3.4 Task D. The ratings of each material or materials system from Tasks B and C are contained in Chapter 15 along with the recommendations for future detailed development and refinement work on these materials.

Table 1.1

Zero Maintenance Candidate Materials Categories

Category 1: Fiber Concrete, Polymer Concrete, and Expansive-Cement Concrete:

- Fiber-Reinforced Concrete
- Polymer Concrete
- Polymer-Portland Cement Concrete
- Polymer-Impregnated Concrete
- Expansive-Cement Concretes

Category 2: Ceramic Materials:

- Calcined Bauxites
- Refractory Shapes
- Cement Clinker Aggregates
- Expanded Clay, Shale, and Slate as Lightweight Aggregates
- Some Pozzolans, Especially Fly Ash, and Calcined Shale
- Slag, Particularly Finely Ground Granulated Slag as a Constituent of Portland-Blast-Furnace Slag Cement

Category 3: Sealant Systems:

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

(Continued)

Table 1.1 (Concluded)

Category 4: Noncalcareous Inorganic Cements:

- Sulfur
- Phosphate Cements
- Sorel Cements

Category 5: Special Construction Techniques:

- Precast Concrete
- Prestressed Concrete
- Vacuum Processed Concrete

Category 6: Organic Cementing Systems:

- Gussasphalt (Mastic Asphalt)
- Asbestos Asphalt
- Rubberized Asphalt
- Epoxy
- Fiber-Reinforced Asphalt Mixtures
- Sulfur-Modified Asphalt Mixtures

CHAPTER 2

ASPHALT CONCRETE

INTRODUCTION

2.1 BACKGROUND

The information presented in this chapter provides a capsule look at the current state-of-knowledge of asphaltic concrete as it is used today. This material is not a zero-maintenance candidate material as experience gained from its continuous use from the 19th century has indicated a need for constantly routine maintenance. The information presented in this chapter is to be used as a comparison base for other types of improved asphalt systems, some of which are described in Chapters 3, 4, and 5.

Since the beginning of placing asphalt pavements in the United States in the 19th century,[1] many different types of asphalt paving mixtures have been developed. One such mixture is asphaltic concrete. Asphaltic concretes are generally preferable to sheet asphalt, sand asphalt, or stone-filled sheet asphalt when the pavement is subjected to heavy traffic.[2] The physical properties of conventional asphalt concretes and the influence of various parameters such as binder contents, temperatures, aggregates, etc., are presented in the following sections. It should be noted that some of the data presented are not normally used to evaluate asphalt concrete; therefore, such data should not be weighed heavily.

2.2 DESCRIPTION OF MATERIAL

Asphalt concrete (conventional asphalt concrete) is a "high quality, thoroughly controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass..."[3]

2.3 TEMPERATURE AND TIME DEPENDENCY

A review of literature[4,5,6,7] indicates that the stress-strain characteristics of an asphalt are temperature dependent and time dependent. Any change in test temperature or loading rate (time) can produce large changes in the strength characteristics.

2.4 COMPRESSIVE STRENGTH

Compression testing is not a method normally used for evaluating asphalt concretes. Methods such as the Marshall stability test are generally used to evaluate asphalt concretes; therefore, compressive strength data are very limited and no real significance should be placed on it.

Hewitt and Slate[7] conducted triaxial compression tests on 2-1/2- x 2-1/2- x 5-in. specimens using confinement pressures of 0 psi and 30 psi. Specimens were prepared using one type aggregate, one asphalt content (5.5 percent) and five types of asphalt. Test temperatures of 80F and 140F were used at a load rate of 0.05 in./min. The unconfined test yielded compression values of 35 to 48 psi at 140F and 100 to 155 psi at 80F (See Table 2.1). The confined triaxial test values ranged from 199 to 205 psi at 140F and 244 to 285 psi at 80F. The effect of the different types of asphalt on the compressive strength did not appear to be very significant, but this may be due in part to the relatively narrow range of asphalt penetrations (70-94) with the exception of one asphalt. Unconfined compression data by Hickey[8] in which a different asphalt concrete (low voids) was tested at a loading rate of 0.2 in. per minute and at 140F indicated a compressive strength range of 108 to 158 psi as the asphalt content was decreased from 8.5 percent to 7.5 percent. These asphalt concrete mixtures had very low void contents (0.18 to 1.15 percent), however, which are lower than most designs allow.

The above unconfined compression testing data indicate that the range or variation in the compression strength, under the test conditions as previously stated, is small, 35 to 285 psi and changes in asphalt type and content are still within the 35 to 285 psi range. This indicates that compressive strength appears to be influenced more by temperature and rate of loading (time) than by asphalt type or content.

2.5 MARSHALL STABILITY

Marshall stability is a measure of the resistance of a mixture to plastic deformation. Typical marshall stability values are shown in Table 2.2. It should be noted that for properly evaluating increases in stability, other marshall properties, such as flow, should also be evaluated. High stability may not be desirable if the flow values are not within desired limits.

2.5.1 Effects of Amount and Type of Binder. Pignataro[9] conducted marshall stability test on 4-in. diameter by 2-1/2-in.-high asphaltic concrete specimens. Five asphalt contents, three asphalts, and one aggregate were subjected to loading at six different temperatures. The results for the marshall test are presented in Table 2.2. Density and void data of the test specimens are presented in Table 2.3. Pignataro's data[9] indicate that stability values increase with increasing asphalt content up to an optimum and then decrease with increasing asphalt content. A decrease in asphalt penetration also brings about an increase in the mixture's stability. Flow, which is always used in conjunction with stability, always increases with an increase in the asphalt content. Density and air void values are also influenced by asphalt content and therefore, are included in marshall design requirements. Density is similar to the stability and asphalt content relationship in that as the asphalt content increases, the density reaches an optimum value. Void content continues to decrease with increasing asphalt contents. The type or penetration of the asphalt used in the marshall tests did not significantly influence any of the test properties except stability.

2.5.2 Effects of Aggregate and Filler Materials. Marshall stability can be increased by using crushed aggregates instead of ungraded natural aggregates. Figure 2.1, (a) and (b),[10] represents the effects on marshall stability and flow, respectively, that can be expected when using crushed aggregates. From Figure 2.1 it appears that the use of a crushed fine aggregate will improve the marshall stability and flow more than the use of a crushed coarse aggregate. Research by Heithaus and Izatt[11] also indicates that angular aggregates will produce higher stability values and that the addition of filler materials will increase the stability up to a point and then the stability will decrease.

2.5.3 Effects of Proportioning and Curing. The effects of proper proportioning are described in Section 2.5.1 and can be seen in Table 2.2. Curing factors, such as wet curing, dry curing, etc., are not generally used in evaluating stability.

2.5.4 Effects of Temperature. Variations in temperature of an asphaltic concrete can produce strength property changes of large magnitudes. Generally lower strength characteristics occur at high temperature and high strength characteristics at low temperatures. The effect of temperature on marshall stability[9] is shown in Figure 2.2. The stability at 176F is approximately 400 lb, but the stability at 86F is approximately 10 times greater. Increases in strength characteristics such as shown in Figure 2.2 are not uncommon for asphaltic concrete. The changes in the compressive strength of unconfined specimens as given in Table 2.1 also show a rapid increase as the temperature is reduced from 140F to 80F. Even higher strength properties can be obtained as the temperature is reduced closer to the glassy temperature range of the asphalt.

2.6 FLEXURAL STRENGTH

2.6.1 Effects of Amount and Type of Binder. Flexural testing,[12,13] indicates that the stiffness and modulus of rupture of asphalt concrete mixtures are influenced by the penetration grade of the asphalt. Figure 2.3 and Table 2.4 show how these flexural properties can be influenced by the different asphalts. Penetration grade asphalt 40/50 when tested at 25F at a loading rate of 0.02 in./min shows a stiffness value of over 90,000 psi and a modulus of rupture of approximately 900 psi. Under the same test conditions, a 200/300 penetration grade asphalt mixture has a stiffness of less than 12,000 psi and a modulus of rupture of approximately 200 psi. Zuehlke[13] has shown (Table 2.5) that variations in the binder content do not produce large changes in the flexural properties, as compared to changes that occur with temperature variation.

2.6.2 Effect of Aggregate and Filler Materials. The aggregates in Table 2.5 were dense graded. Even though two of the aggregates were crushed stone and the other a crushed gravel, all of different gradations, the difference in the resulting flexural strength properties was not significant. The addition of filler tends to decrease the void content and increase density.[14,15] The resulting change in densification results in different mechanical properties.[15]

2.6.3 Effects of Proportioning and Curing. The trends established in Table 2.2 and described in Section 2.5.1 for the effects of proportioning on marshall stability also apply to flexural strength. As in the case for marshall stability curing factors such as wet cure, dry cure, etc., are generally not considered in the evaluation of flexural strength.

2.6.4 Effects of Temperature. Temperature has a great effect on flexural strength properties of asphalt concretes. The flexural stiffness and modulus of rupture are temperature dependent. As seen in Tables 2.4 and 2.5 and Figure 2.3, the stiffness of an asphalt concrete will vary from near zero at high temperatures (100F) to near 400,000 psi at -35F, while the modulus of rupture will also vary from near zero to 1200 psi. Such changes in the strength of an asphalt concrete are not unusual. At low temperatures, the failure of an asphalt concrete is partly in the asphalt and partly in the aggregate, but at high temperatures the failure is in the asphalt cement binder and not in the aggregate.[12]

2.7 TENSILE STRENGTH

2.7.1 Effects of Amount and Type of Binder. Epps and Monismith[4] performed direct tensile tests on various asphaltic concretes at one temperature (68F) and one time of loading (0.1 sec). The results are shown in Table 2.6. The tensile stress at fracture was approximately 600 psi for asphalt concrete prepared from penetration grade 40-50, 60-70, and 85-100 asphalts at an asphalt content of 6 percent. The strain values for the 60-70 and 85-100 penetration grade asphalts were approximately 0.02 in./in., but the 40-50 penetration grade asphalt mixture had a strain of 0.005 in./in.; a significant reduction in strain with a corresponding increase in stiffness. A crushed basalt aggregate was also tested at asphalt contents of 5.7, 6.7, and 8.7 percent (Table 2.6). Regardless of the asphalt content, there was no significant difference between stress, strain, or stiffness at fracture of the test specimen.

2.7.2 Effects of Aggregate. The data in Table 2.6 also show the effects of changing the gradation or type of aggregate. The data for the California "fine" gradation appears to have a slightly higher stress, strain, and stiffness than the "medium" gradation mixtures which had tensile strengths slightly higher than the "coarse" gradation mixtures. This may be partly due to a higher filler or fines content but this is not conclusive. The tensile strengths of asphalt concretes prepared from crushed limestone, crushed gravel, and crushed granite are shown in Figure 2.4. The granite and limestone aggregates appear to follow the same path, but the granite aggregate mixture had a lower stress value (368 psi) than the limestone aggregate mixture (465 psi) and a strain value that was considerably lower. The crushed gravel mixture had a higher stress value than the other two aggregate mixtures, but the strain was approximately the same as the strain in the granite mixtures. Different aggregates will cause some differences in the tensile properties of a mixture, but the effect on the tensile strength will usually not be significant.

2.7.3 Effects of Proportioning and Curing. There is little information on proportioning effects on tensile strength, although it is believed to follow trends similar to those of marshall stability and flexural strength. Again, curing factors such as wet cure, dry cure, etc., are not usually a consideration with asphaltic concretes.

2.7.4 Effects of Temperature. Tensile testings by Hewitt and Slate[7] and Tons and Krokosky,[16] indicate that temperature has a significant influence on tensile properties of an asphalt concrete mixture. The test results from Hewitt and Slate[7] (Table 2.7) show that the tensile strength at 60F can increase more than 10 times when the temperature is reduced to 0F; that is from 20 to 50 psi at 60F to 300 to 500 psi at 0F. The same temperature influence was also found by Tons and Krokosky,[16] as shown in Figure 2.5. At temperatures of 120F the tensile strength was approximately 20 psi, but as the temperature was decreased, the tensile strength increased by 300-500 psi. Somewhere between +20F and -20F the asphalt was believed to have approached the glassy temperature range, and as a result the binder became more glass-like than elastic.[16] As a result, the tensile strength was slightly reduced. The effects of loading rates (time) on tensile strength can also be seen in Table 2.7. Faster loading results in higher strength and slower loading produces lower strength characteristics.

2.8 DIMENSIONAL CHANGES

2.8.1 Thermal Expansion and Contraction. Littlefield[17] evaluated 3-in.- by 3-in.- by 16-in.-long asphalt concrete beams and concluded that:

a. Asphalt binder from different sources will produce different amounts of expansion and contraction between 0F and 130F.

b. The thermal coefficient of expansion data should include the temperature range, grade, and source of the asphalt binder.

c. The amount of shrinkage during expansion and contraction testing was more than the amount of expansion; consequently, the mixture tended to densify.

The expansion and contraction characteristics obtained by Littlefield[17] are presented in Table 2.8. Other research[18] indicates that an asphalt concrete exhibits two different thermal coefficients of expansion and contraction. One is called the solid thermal coefficient and applies to the expansion and contraction measure below the transition temperature of the asphalt. Above the transition temperature the coefficient is called a fluid thermal coefficient.

2.8.2 Due to Moisture Changes. Test conducted by Zube and Cechetini[19] on asphalt concrete made with highly absorptive aggregates indicated that moisture absorbed from the air can cause considerable expansion. When exposed to moist air the test specimens expanded and when exposed to drier air the specimens contracted, but the contraction was always less than the expansion. This expansion and contraction is shown in Figure 2.6. Such expansion and contraction can cause rapid failure or cracking of the asphalt concrete. Asphalt concrete made with nonabsorptive aggregate did not show this type of behavior as a result of being exposed to moisture.

2.9 DURABILITY

Asphaltic concrete durability is not a measurable physical property; therefore data are not available for this property.

2.10 ABRASION RESISTANCE

Abrasion tests are not usually performed on asphalt concrete mixtures, hence, no abrasion resistance data are presented.

2.11 TOUGHNESS

Toughness is another physical property that is not normally used in evaluating asphalt concretes. Data, if available, would be dependent on temperature and time.

2.12 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

A review of the literature[20,21,22,23] indicates that the fatigue characteristics of bituminous concrete are influenced by parameters such as binder content, void content, temperature, etc., but the degree of influence will vary because of the various testing conditions and methods used by researchers. Mixtures tested under controlled stress (load) conditions will have different fatigue properties than the same mixture when tested under a controlled strain (deflection) loading mode (Figure 2.7). Epps and Monismith[23] summarized some of the existing fatigue data to show the influence of various parameters on fatigue of bituminous mixtures. Tables 2.9, 2.10, and 2.11 show the effect of many of the various parameters.

2.13 PERMEABILITY

Tests by Hickey,[8] Goode and Lufsey,[24] and Mullen[25] indicate that the permeability of an asphalt concrete decreases as the air void content is decreased. They concluded that generally asphalt concretes with low air voids (3 to 6 percent) are impermeable to either water or air. Air permeability data[24] are presented in Table 2.12 and show the effect of void content. Water permeability tests[25] indicated that the asphalt concrete specimens with less than 6 percent voids had a permeability of 0.0001×10^{-4} cm/sec or less. Specimens with a permeability of 0.0001×10^{-4} cm/sec or less are considered to be impermeable. Since the permeability of an asphalt concrete is dependent on the amount of air voids, the permeability can be controlled by varying the amount of binder in the mixture or by changing aggregate gradations.

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CHAPTER 2

ASPHALT CONCRETE

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Table 2.1

Summary of Asphaltic Concrete Compressive Strength Data as Determined by Triaxial Testing[7]

	Strength at 0.05 in./min Rate of Loading, psi									
	80F					140F				
	Asphalt Penetration at 77F					Asphalt Penetration at 77F				
Property	70	94	173	80	90	70	94	173	80	90
Unconfined compression, psi	155	138	100	130	123	48	41.8	36.0	35.1	42.2
Triaxial compression, psi (30 psi lateral)	285	267	244	265	260	205	203	197	213	199
Modulus of elasticity, psi unconfined	10,590	10,480	7,960	9,540	9,505	3,520	3,314	2,787	2,810	3,480
Triaxial (30 psi lateral)	17,500	18,670	13,930	15,810	15,650	7,470	10,600	8,025	8,820	9,030

Table 2.2

Summary of Asphaltic Concrete Marshall Stability and Flow Values for One Aggregate[9]

% Asphalt cement	Stability, lb Penetration 52	Stability, lb Penetration 76	Stability, lb Penetration 108	Flow, 0.01 in. Penetration 52	Flow, 0.01 in. Penetration 76	Flow, 0.01 in. Penetration 108
Test temp., 80 C						
5	323	290	260	8.5	8.0	8.0
5.5	374	340	312	9.5	9.0	8.5
6	419	380	352	11.0	10.5	10.0
6.5	410	360	340	13.5	12.5	11.5
7	402	330	290	17.5	16.0	14.5
Test temp., 70 C						
5	566	472	425	9.0	8.5	8.0
5.5	618	537	500	11.0	10.5	9.5
6	672	587	560	13.5	12.5	11.5
6.5	615	581	538	15.5	14.5	13.5
7	600	531	476	20.0	19.0	18.0
Test temp., 60 C						
5	861	720	615	11.0	10.0	9.5
5.5	1010	900	773	12.0	11.5	11.5
6	1060	941	850	15.0	14.0	13.5
6.5	1040	910	821	18.0	17.0	15.5
7	970	852	750	22.0	21.0	18.5
Test temp., 50 C						
5	1400	1211	985	11.0	10.0	9.5
5.5	1670	1462	1180	12.5	12.0	11.5
6	1770	1548	1250	15.5	14.5	14.0
6.5	1710	1410	1205	18.5	17.5	16.5
7	1500	1340	1050	22.0	21.5	20.0

(Continued)

Table 2.2 (Concluded)

% Asphalt cement	Stability, lb			Flow, 0.01 in.		
	Penetration 52	Penetration 76	Penetration 108	Penetration 52	Penetration 76	Penetration 108
Test temp., 104F						
5	2180	1990	1450	12.0	12.5	13.0
5.5	2610	2390	1847	14.0	14.5	15.0
6	2800	2500	2004	16.0	17.0	17.5
6.5	2600	2382	1920	18.5	19.0	20.0
7	2300	2000	1600	22.5	23.0	23.5
Test temp., 86F						
5	3710	2860	2400	12.5	13.0	14.0
5.5	4200	3310	2852	14.5	15.5	16.0
6	4440	3610	3100	16.5	17.5	18.5
6.5	4250	3400	2945	19.0	20.0	21.5
7	3860	2995	2490	23.0	23.5	24.5

Table 2.3

Summary of Density-Voids Data for Asphaltic Concrete[9]

% Asphalt Cement by Weight of Total Mix	% Voids of Total Mix	% Total Voids Filled with Asphalt	Density, pcf
Asphalt penetration 52			
5	6.6	64.0	149.5
5-1/2	4.5	74.5	151.7
6	2.8	83.7	153.1
6-1/2	1.8	90.0	153.6
7	1.2	93.5	153.3
Asphalt penetration 76			
5	6.6	64.2	149.4
5-1/2	4.5	74.4	151.5
6	2.9	83.5	152.9
6-1/2	1.8	89.7	153.4
7	1.3	92.9	153.0
Asphalt penetration 108			
5	6.6	63.9	149.3
5-1/2	4.6	74.3	151.4
6	3.1	82.4	152.6
6-1/2	1.8	89.8	153.4
7	1.3	93.2	153.1

Table 2.4

Average of Asphaltic Concrete Flexure Test Results[12]

Asphalt Grade Penetration	Temperature, °F	Stiffness Modulus, psi	Rupture Modulus, psi
40/50	+25	91,318	891
	-5	283,109	827
	-35	420,000	779
85/100	+25	48,214	508
	-5	226,826	905
	-35	319,077	893
200/300	+25	11,671	225
	-5	117,482	895
	-35	268,118	869

Table 2.5

Effect of Aggregate on Flexural Strength of Asphaltic Concrete[13]

Temperature, °F	Asphalt Content, % by Wt	Bulk Density, pcf	Voids, %	Maximum Center Load, lb	Stiffness, lb/in.	Modulus of Rupture, psi
(a) Crushed Gravel Surface Course Mixture						
40	5.7	146.1	5.3	1,100	3,280	1,030
	7.7	146.5	2.2	1,320	2,120	1,240
100	5.7	146.3	5.2	4	10	4
	7.7	145.3	3.0	14	18	13
(b) Crushed Stone Surface Course Mixture						
40	5.5	148.8	6.3	1,400	4,000	1,000
	7.5	149.0	4.0	1,250	2,600	1,170
100	5.5	148.5	6.5	21	96	20
	7.5	149.3	3.8	20	20	19
(c) Crushed Stone Binder Course Mixture						
40	5.2	149.9	5.6	1,280	4,400	1,200
	7.2	150.1	2.1	1,040	2,200	970
100	5.2	150.6	5.2	20	48	19
	7.2	150.5	1.8	14	20	13

Table 2.6

Direct Tensile Test Results for Asphaltic Concrete[4]

Mixture	Asphalt Penetration	Asphalt Content %	Air Voids %	Stress at Fracture psi	Strain at Fracture × 10^{-6}	Stiffness at Fracture × 10^{-3} psi
British Standard 594 G.A.*	40-50	7.9	5.22	654	916	714
Calif. Coarse, G.A.	85-100	6.0	6.00	479	1593	301
Calif. Fine, G.A.	85-100	6.0	5.51	775	2100	369
Calif. Medium, G.A.	85-100	6.0	4.41	596	2027	294
Calif. Medium, G.A.	60-70	6.0	5.33	626	2054	305
Calif. Medium, G.A.	40-50	6.0	4.69	611	505	1210
Calif. Medium, B.A.*	60-70	5.7	8.10	470	1196	394
Calif. Medium, B.A.	60-70	6.7	6.57	513	1112	461
Calif. Medium, B.A.	60-70	8.7	1.61	560	1682	333
Calif. Medium, G.A.	85-100	5.2	6.82	368	971	379
Calif. Medium, L.A.*	85-100	4.7	6.80	465	1427	326
Calif. Medium, C.G.*	85-100	4.6	7.55	523	1044	502

* G.A. Crushed granite aggregate.
B.A. Crushed basalt aggregate.
L.A. Crushed limestone aggregate.
C.G. Crushed gravel aggregate.

Table 2.7

Tensile Test Results of Asphaltic Concrete as Influenced by Temperature and Rate of Loading

Temperature, °F	Rate of Loading in./min	Tensile Strength, psi				
		70*	94*	173*	80*	90*
60	0.002	45.9	22.6	13.2	38.9	18.7
40	0.002	92.5	62.3	36.1	117	47.8
	0.006	-	-	-	-	71.2
	0.02	-	-	-	-	152
	0.05	170	131	81.4	189	198
	0.5	450	373	246	349	520
20	0.002	280	210	118	316	286
0	0.002	487	465	313	467	502

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*Asphalt cement penetration at 77F.

Table 2.8

Expansion and Contraction Characteristics of Asphaltic Concrete[17]

A.C. Source	Type Asphalt Penetration Grade	Expansion 0F to 100F in. per 100 Ft	Shrinkage 130F to 70F in. per 100 Ft	Shrinkage 70F to 0F in. per 100 Ft	Coefficient of Thermal Expansion °F × 10^{-5}
E	120-150	0.99	0.48	0.92	1.33
F	120-150	1.22	0.48		1.41
C	120-150	1.37	0.51		1.44
B	120-150	1.38	0.54		1.46
A	120-150	1.42	0.59	1.12	1.48
E	85-100	0.88	0.59	0.83	1.32
F	85-100	1.07	0.52		1.37
C	85-100	1.20	0.64		1.43
A	85-100	1.25	0.49		1.43
D	85-100	1.39	0.59		1.43
B	85-100	1.56	0.68	1.19	1.52
F	60-70	1.18	0.59	0.93	1.33
C	60-70	1.29	0.65		1.40
E	60-70	1.46	0.62		1.45
A	60-70	1.49	0.66		1.61
B	60-70	1.67	0.64	1.32	1.63

Table 2.9

Factors Affecting the Stiffness and Fatigue Behavior of Asphaltic Concrete Mixtures[23]

Factor	Change in Factor	Effect of Change in Factor: On Stiffness	Effect of Change in Factor: On Fatigue Life in Controlled Stress Mode of Test	Effect of Change in Factor: On Fatigue Life in Controlled Strain Mode of Test
Asphalt penetration	decrease	increase	increase	decrease
Asphalt content	increase	increase[a]	increase[a]	increase[b]
Aggregate type	increase roughness and angularity	increase	increase	decrease
Aggregate gradation	open to dense gradation	increase	increase	decrease[d]
Air void content	decrease	increase	increase	increase[d]
Temperature	decrease	increase[c]	increase	decrease

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a. Reaches optimum at level above that required by stability considerations.
b. No significant amount of data; conflicting conditions of increase in stiffness and reduction of strain in asphalt make this speculative.
c. Approaches upper limit at temperature below freezing.
d. No significant amount of data.

Table 2.10

Selected Results from Controlled Stress Tests of Asphaltic Concrete[23]

Variable	Change in Variable	Change in Fatigue Life[a]
Asphalt penetration	92-33	500,000 to 1,000,000
	120-60	6,000 to 250,000
	110-40	no significant change[b]
	180-40	250,000 to 1,000,000
	85-13	no significant change
Asphalt content[c]	5.3-6.7%	2,000 to 20,000
	6.0-7.5%	6,000 to 40,000
	3.5-6.5%	6,000 to 2,500,000
Aggregate type	smooth to rough surface texture	800,000 to 1,000,000
		8,000 to 40,000
		750,000 to 1,000,000
Aggregate gradation	coarse to fine	450,000 to 1,000,000
		450,000 to 1,000,000[d]
		150,000 to 1,000,000[d]
		350,000 to 1,000,000[d]
	addition of filler 0 to 9%	700,000 to 2,500,000
Air void content		25,000 to 125,000 (sandsheet)
		250 to 15,000 (dense graded)
	10-3%	250 to 5,000 (dense graded)
		30,000 to 300,000 (sandsheet)
	9-4.5%	300,000 to 1,000,000 (sandsheet)
Temperature		200,000 to 1,000,000 (Avg. Values)
	40-68 F	600,000 to 1,000,000 (Avg. Values)
	32-86 F	no significant change[b]
	14-50 F	no significant change

a. Comparisons based on results from stress-fatigue life relationships.
b. Little difference was noted on ε-N plot provided the mixture did not exhibit nonlinear behavior.
c. Optimum asphalt content used to establish maximum fatigue life.
d. Comparison of sheet asphalt mixtures with dense graded mixtures.

Table 2.11

Selected Results from Controlled Strain Tests of Asphaltic Concrete[23]

Variable	Change in Variable	Change in Fatigue Life[a]
Asphalt penetration	114-47	100,000 to 9,000
	200-50	no significant change
Asphalt content		no significant data available
Aggregate type	smooth to rough surface texture	no significant change 300,000 to 1,000,000
	rounded to angular shape	significant change
Aggregate gradation	coarse to fine	no significant change limited data available
Air void content	9-5%	2,000 to 3,500 (dense graded)
Temperature		2,000 to 1,000,000
	40-75 F	300,000 to 1,000,000
	23-104 F	100 to 1,000,000

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[a] Comparisons based on results from strain fatigue life relationships.

Table 2.12

Air Permeability Measurements from Asphaltic Concrete[24]

Air Voids %	Aggregate Blend	Permeability per cm^2 $\times 10^{-3}$ cm^3/sec*
4	A	3
	C	5
	E	7
7	A	40
	C	67
	E	115
10	A	98
	C	230
	E	800

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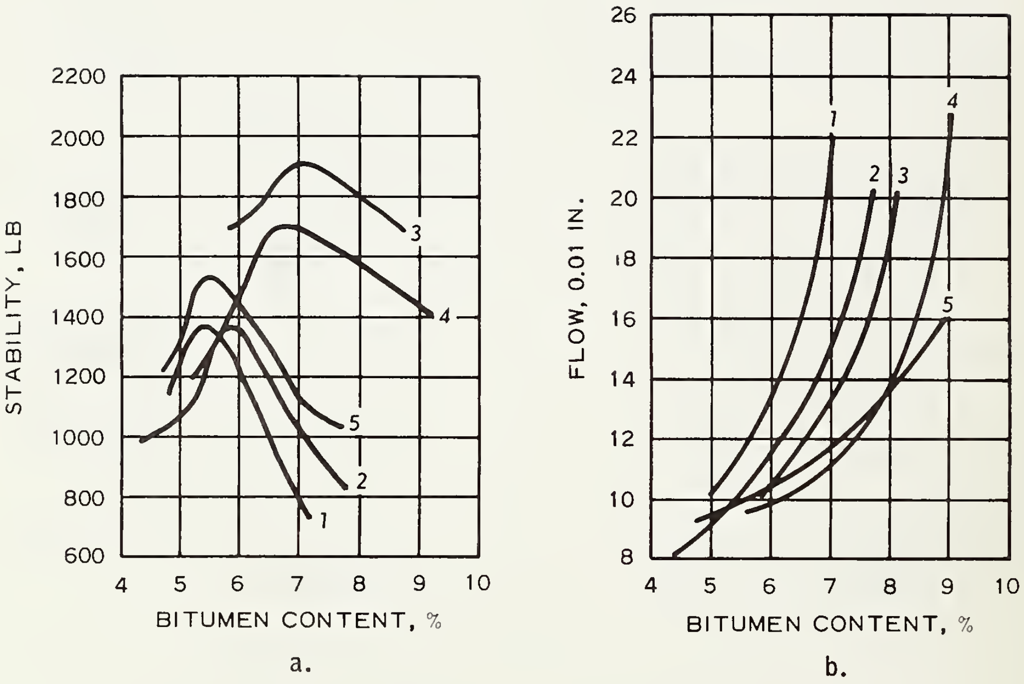
* Vertical edges sealed with paraffin.



a.

b.

COMPOSITION OF MIXTURES

MIXTURE	TYPE OF FINE AGGREGATE (−10#)	TYPE OF COARSE AGGREGATE (+10#)
1	GRADED NATURAL SAND	NATURAL GRAVEL
2	UNGRADED NATURAL SAND	NATURAL GRAVEL
3	CRUSHED SAND	CRUSHED AGGREGATE
4	CRUSHED SAND	NATURAL AGGREGATE
5	GRADED NATURAL SAND	CRUSHED AGGREGATE

Figure 2.1. Marshall stability and flow of asphalt concrete[10]

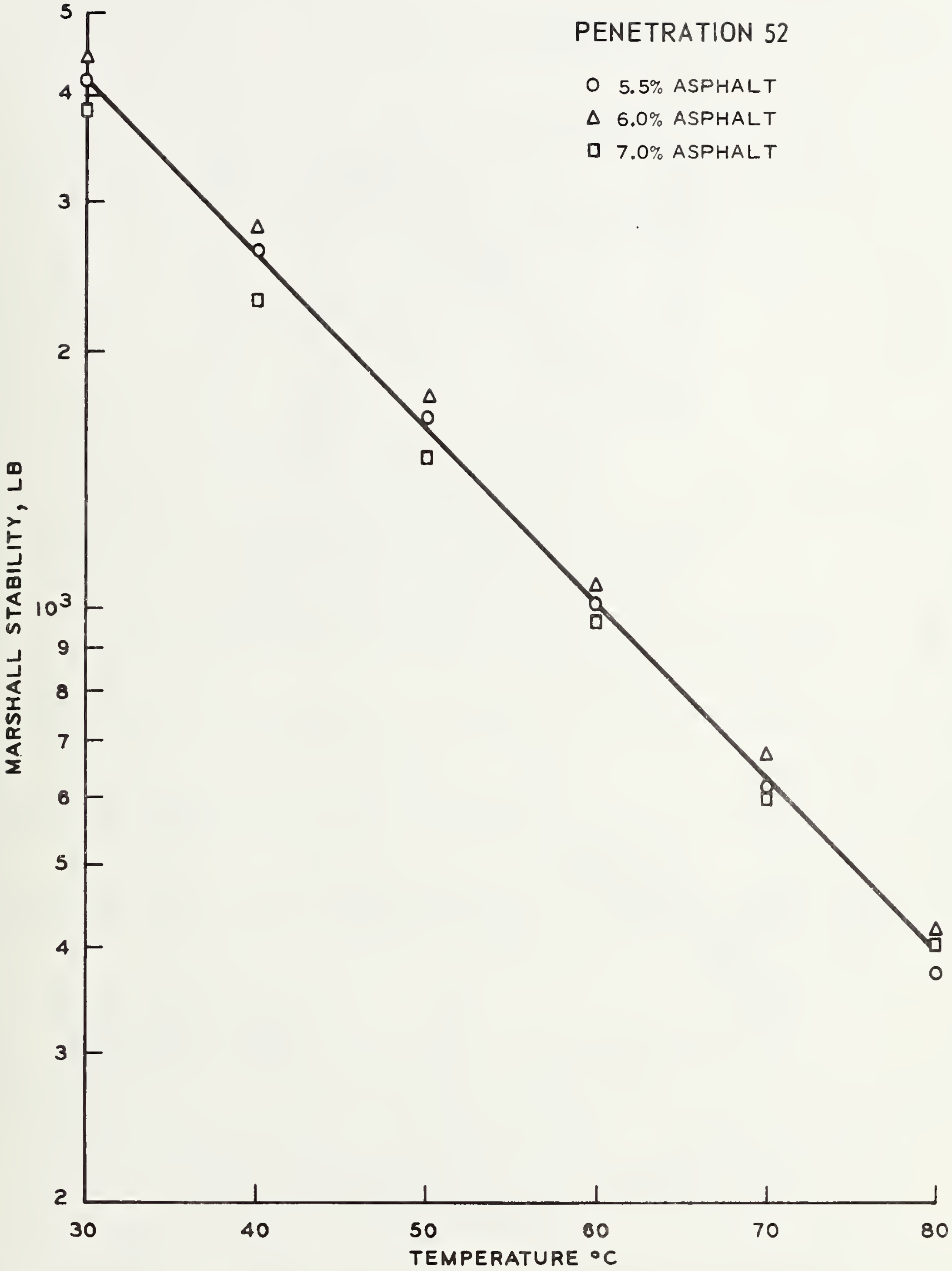


Figure 2.2. Marshall stability as affected by temperature[9]

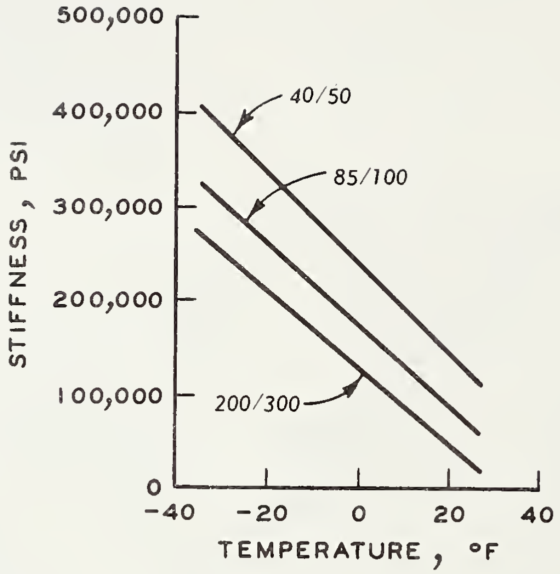


Figure 2.3. Effect of temperature on stiffness of asphaltic concrete[12]

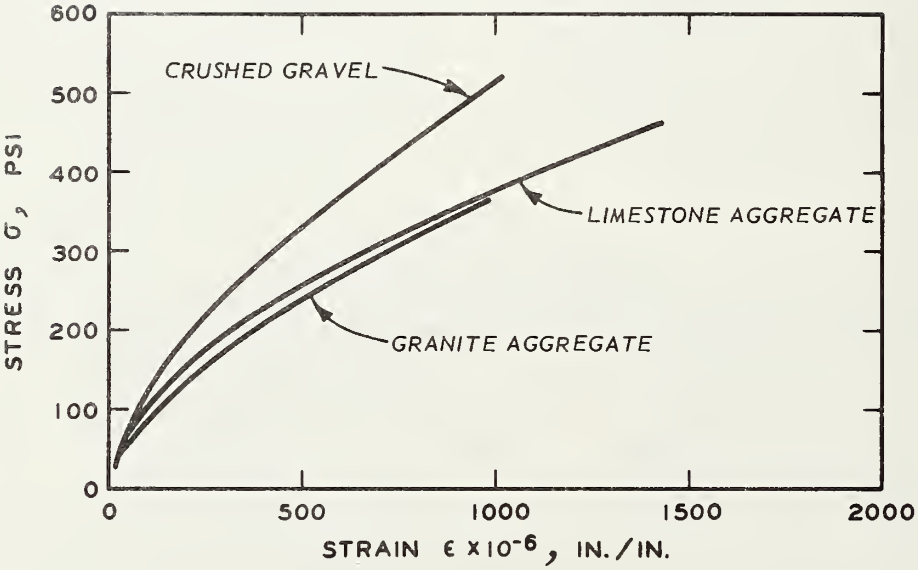


Figure 2.4. Effect of aggregate type on stress-strain behavior of California medium graded mixes; 85-100 penetration asphalt--direct tension tests[4]

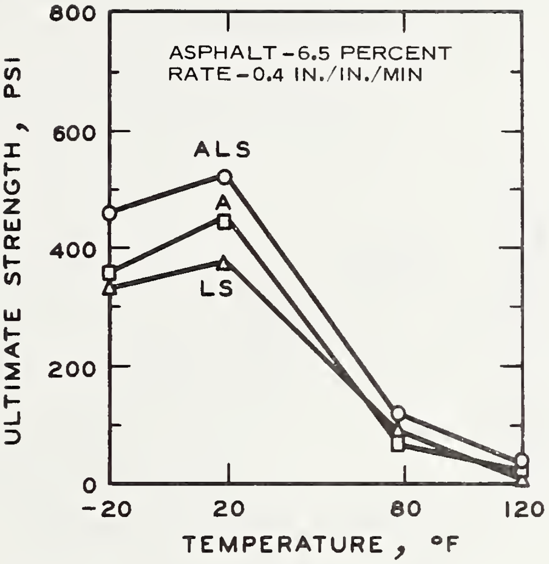


Figure 2.5. Effect of temperature on tensile strength of asphalt concretes containing: Limestone Filler (LS), Asbestos Fiber (A), and a combination of Asbestos and Limestone (ALS)[16]

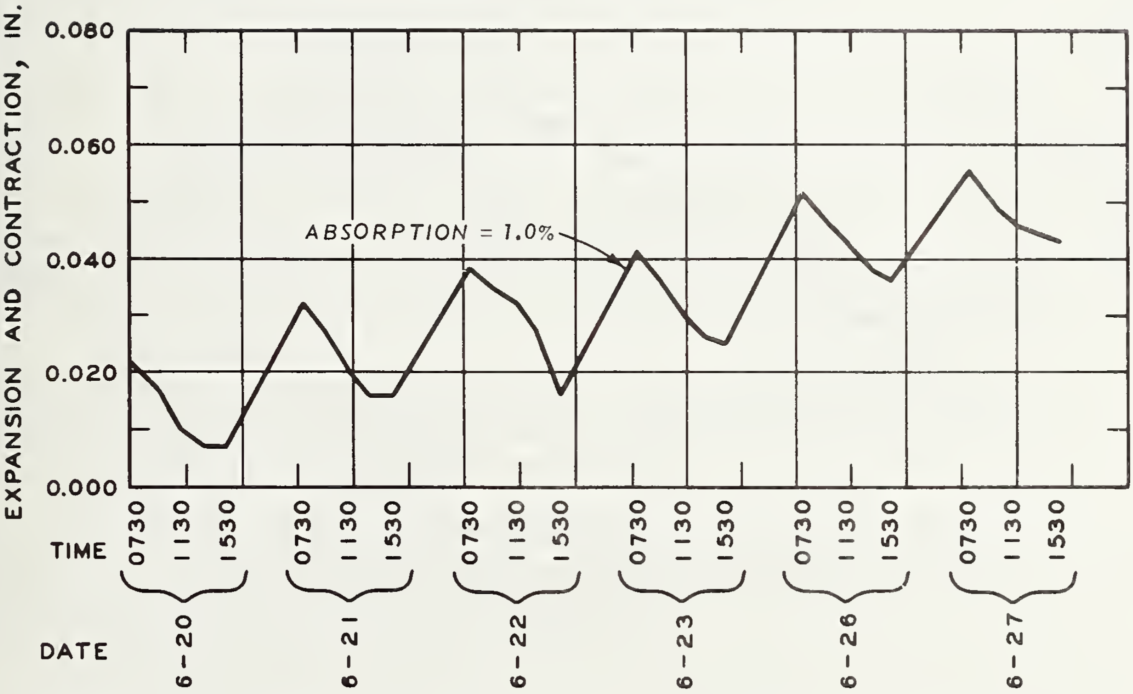
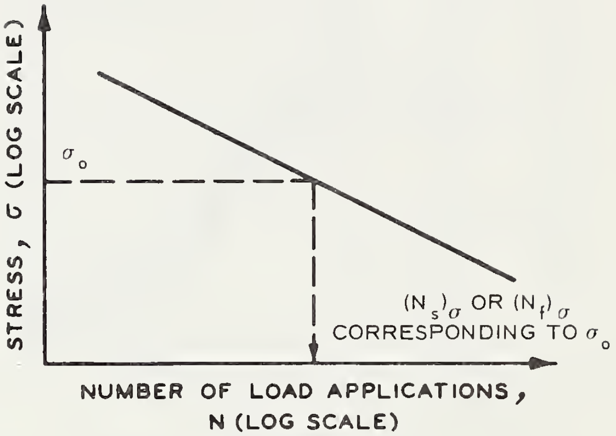
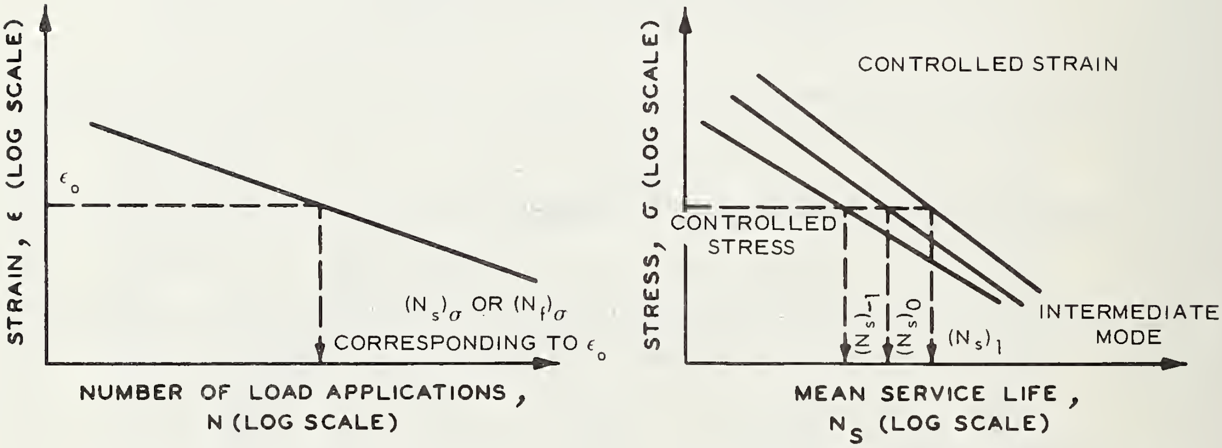


Figure 2.6. Expansion and contraction of asphaltic concrete[19]



a. STRESS VS LOAD APPLICATIONS
(CONTROLLED-STRESS)



b. STRAIN VS LOAD APPLICATIONS
(CONTROLLED-STRAIN)

c. INITIAL STRAIN VS LOAD APPLICATIONS,
COMPARISON FOR CONTROLLED-STRESS
AND CONTROLLED-STRAIN TESTS

Figure 2.7. Fatigue diagrams for asphaltic concretes[23]

CHAPTER 3

GUSSASPHALT

INTRODUCTION

3.1 BACKGROUND

The use of gussasphalt dates back to the early nineteenth century, but it was not extensively used in pavement construction until 1953 when it was machine-laid for the first time. Gussasphalt is used as a surface wearing course and is placed in thin layers of approximately 3.5 cm (1.4 in.). Gussasphalt, which provides a tough, durable, and watertight pavement surface, is primarily used where heavy traffic is anticipated.[1] Most construction involving gussasphalt pavements has been done in Europe.[2] Only a limited amount of gussasphaltic pavements has been placed in this country. The Pennsylvania Department of Transportation and the Pennsylvania Turnpike Commission placed small test sections of gussasphalt in 1972.[3]

3.2 DESCRIPTION OF MATERIAL

Gussasphalt is a mixture of a relatively hard asphalt, mineral filler, aggregate, and sometimes Trinidad Lake Asphalt. These materials are proportioned to produce an essentially voidless mixture which can be cast or poured in place at temperatures of 374 to 437F.[3.4] The material proportions of a typical gussasphalt mixture are shown in Tables 3.1, 3.2, and 3.3.

MECHANICAL PROPERTIES

Some of the various gussasphalt mixture characteristics are presented in the following paragraphs. The amount of available data on the various characteristics is limited.

3.3 COMPRESSIVE STRENGTH

3.3.1 Effects of Amount and Type of Binder. A relatively hard asphalt cement with a 45 penetration is commonly used in gussasphalt mixtures, although the general range of penetration may vary from 15 to 60. The amount of asphalt binder is usually within the range of 6.5 to 9.0 percent, based on the weight of the total mixture.[2] Data relating to the effects of various amounts and types of binder on the compressive strength of gussasphalt mixtures were not available.

3.3.2 Effects of Aggregate, Filler, and Fiber Materials. Data on the effects of different aggregates and aggregate gradations were not available. The gradation as given in Table 3.3 is typical, but coarse aggregate as large as 1/2 in. can be used. One requirement in the aggregate gradation is that a high percentage of filler be used. Schultz and Jones[1] investigated the effects of the filler on the strength of gussasphalt mixtures. Figure 3.1 shows the results of varying the type of filler material. Normally, a limestone filler is used in gussasphalt, but in Figure 3.1, 5 percent of the limestone filler was substituted with asbestos in one test and with fiberglass in the other test. The effects of using different fillers (5 percent of a total filler content of 25.1 percent) were not significant. The most significant change in compressive strength is associated with the change in temperature, which is discussed in Section 3.3.4.

3.3.3 Effects of Proportioning and Curing. Generally, the proportions as given in Table 3.1 are within an acceptable limit. The amount of mineral filler that is used is very seldom below 20 percent, and the amount of binder used is very seldom above 10 percent.[2] Data with respect of the effect of proportioning on the compressive strength were not available nor was information on curing.

3.3.4 Effects of Temperature. Gussasphalt reacts to temperature changes the same way that conventional asphalt mixtures react, that is, as the temperature decreases, the compressive strength increases. Figure 3.1 shows the effect of temperature on the compressive strength of gussasphalt and conventional asphalt mixtures. At 120F, the compressive strength of gussasphalt is in the range of 300 to 400 psi, but at 32F, the compressive strength increases to 1300 to 1400 psi. The gussasphalt mixtures are not as sensitive to temperature changes as conventional asphalt concretes. At temperatures above 100F, gussasphalt shows compressive strength that is twice as great as conventional asphalt concretes at the same temperature, but at temperatures below 77F, the conventional asphalt mixtures show greater compressive strength. There is a transition zone between 77 and 90F in which the compressive strength of gussasphalt and conventional asphalt concrete is approximately the same. The rate of change in compressive strength for gussasphalt is not as rapid as is the rate of change associated with a conventional asphalt concrete.[1]

3.4 FLEXURAL STRENGTH

Information and data relating to the flexural strength of gussasphalt were not available.

3.5 TENSILE STRENGTH

3.5.1 Effects of Amount and Type of Binder. Data relating to the effects of different amounts and types of gussasphalt binder on tensile strength were not available.

3.5.2 Effects of Aggregate, Filler, and Fiber Materials. There is information that a change in the aggregate requirements can produce an improvement in the tensile properties,[1] but this is not substantiated by any available data. The effects of an asbestos, fiberglass, and limestone filler on tensile strength are shown in Figures 3.2, 3.3, and 3.4. As previously stated, limestone filler is the most common filler material. In the tensile testing, 5 percent of the limestone filler was replaced with 5 percent asbestos or fiberglass. At temperatures above 60F, the gussasphalt containing all limestone filler showed the highest tensile strength, but near 32F the mixture containing asbestos or fiberglass filler showed superior strength over the mixture with limestone filler. The gussasphalt mixture containing the fiberglass filler showed the highest tensile strength at 32F. In contrast to a conventional asphalt mixture containing 2-1/2 percent limestone filler and 2-1/2 percent asbestos filler, gussasphalt has a higher tensile strength.

very brittle and being chipped off by the studded tire during wear testing. Additional tests would be necessary to determine the effects of binder on the abrasion resistance of gussasphalt.

The results of wear testing of gussasphaltic surface is shown in Figure 3.5. Figure 3.6 shows the results of wear testing on conventional asphalt surfaces. Table 3.4 compares the results of the wear of gussasphalt to the wear of a conventional asphalt surface.

3.8.2 Effects of Aggregate and Filler. In the wear testing conducted by Kohler and Tappert,[6] the wear of the gussasphalt surface was found to be generally independent of the type of aggregate used in the gussasphalt. Figure 3.5 shows this aggregate independence. In most cases, the wear of the gussasphalt pavement is approximately the same regardless of type or hardness of the aggregate.

3.8.3 Effects of Proportioning and Curing. No information on the effects of proportioning on abrasion resistance could be found. The only data regarding the effects of curing were from testing conducted by Kohler and Tappert.[6] Prior to testing, specimens of gussasphalt and asphaltic concrete were subject to either a preliminary treatment of 10 days of air storage at 50F under wet conditions or to 25 freeze-thaw changes (5F to 68F). Figures 3.5 and 3.6 show the results of these preliminary treatments. The wear on the gussasphaltic surface was found to be relatively independent of the preliminary treatments, but the wear of the open graded asphaltic concrete was found to be influenced by the preliminary treatments.[6]

3.8.4 Effects of Temperature. Figure 3.5 shows the effects of low temperature on the abrasion resistance (wear) of a gussasphaltic pavement. At 50F,wear in the amounts of 30 to 35 g/cm^2 might be expected. As the temperature drops to 32F, even less wear can be expected, but at 14F there is a slight increase in the amount of wear on the gussasphaltic pavement.[6] A reason for the increase in wear at 14F was presented in Section 3.8.1. A wear ratio of gussasphalt versus asphaltic concrete is shown in Table 3.4. At temperatures above 32F the asphaltic concrete wears twice as much as the gussasphalt mixture. Only at 14F is the wear ratio the same. Data regarding the effect of temperatures higher than 50F were not available.

3.9 TOUGHNESS

Information on the toughness of gussasphalt was not available.

3.10 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

Information on the fatigue strength and dynamic response of gussasphalt was not available.

3.11 PERMEABILITY

Because gussasphalt is a voidless mixture of aggregate, filler, and binder, it should be impermeable. A field test performed on a test section in Pennsylvania[3] showed that the gussasphalt was impermeable. The permeability test was accomplished by sandwiching wetted gussasphalt between two metal plates and measuring the resistance between the plates. The ohmeter readings were very high (one million to infinity) which indicates that no water had passed through the gussasphalt.[3]

USE OF GUSSASPHALT

3.12 EASE OF CONSTRUCTION

The construction of a gussasphalt pavement is very similar to the construction of a portland cement concrete pavement. Both gussasphalt and portland cement concrete are poured or cast in place. Because the gussasphalt mixture is voidless and is placed in a fluid-like condition, compaction of the mixture is not required.

A special paving train is used to place the gussasphalt. The paving train, which is mounted on longitudinal rails, consists of a spreader with a strike-off bar, a chip spreader, and a chip roller.[2,4] After the gussasphalt has been spread, precoated chips are spread over the hot gussasphalt and lightly rolled into the surface. These chips are used to improve the skid resistance of the gussasphalt.

The speed of paving generally averages 4 to 5 ft per minute. Paving speeds higher than 6.5 ft per minute are not recommended because of the danger of trapping bubbles in the layer of gussasphalt.[4] Under the best conditions and with trained construction crews, output as high as 240 tons per hour can be obtained, but normally, the output of a gussasphalt plant is only 70 to 80 percent of the output of an asphaltic concrete mixing plant. In Germany, 80 to 100 tons per hour plants are common.[4]

There are several problems associated with the placement of gussasphalt mixtures. One such problem is safety. Because the gussasphalt is placed at near 400F, extra precautions must be taken to ensure the safety of all personnel involved in the construction of a gussasphalt pavement. Another problem is running of the gussasphalt on grades. When placed on grades greater than 5 percent, gussasphalt may tend to run.[7]

The gussasphalt must be allowed to cool and take a "set" before traffic can be allowed on the pavement. Sometimes it may be necessary to remove traffic for as long as 24 hours,[7] but some researchers[1] contend that the gussasphalt pavement can be opened to traffic immediately after the gussasphalt cools.

As previously stated, most of the gussasphaltic paving has been done in Europe, and only a limited amount has been placed in this country. The Pennsylvania Department of Transportation placed 1.7 miles of gussasphalt in September 1972 in Dauphin County on TR 422-322 between Eisenhower Interchange and Hummelston. The gussasphalt was placed between stations 275+42 and 321+75 in the eastbound lane and between stations 323+72 and 363+85 in the westbound lane.[3]

The Pennsylvania Turnpike Commission also placed a five-mile test section of gussasphalt west of Bedford, Pennsylvania, in 1972.[7] The test section was placed by the same contractor that placed the test sections for the Pennsylvania Department of Transportation. Information regarding the amount of traffic that used the test section was not given, but the test section is subject to heavy high-speed traffic and severe winters.[7]

3.13 EASE OF MAINTENANCE

Gussasphalt is described as a paving material that is virtually maintenance free.[2] But if maintenance is required, new gussasphalt can be easily poured in place and opened to traffic after it has cooled.[1,8]

3.14 ENVIRONMENTAL COMPATIBILITY

The compatibility of gussasphalt with the environment should be on the same level as any conventional asphalt concrete paving mixture.

3.15 AVAILABILITY AND COST

The availability of gussasphalt in the United States is extremely limited. The special equipment necessary for transporting and placing gussasphalt is not manufactured or readily available in this country. The Pennsylvania Department of Transportation imported most of the equipment necessary to place their test sections. The experience necessary to design and construct a good gussasphaltic pavement is also very limited. In Germany, where the construction equipment is available, the cost of placing gussasphalt is twice the cost of conventional asphalt concrete.[2] In order to place gussasphalt in the United States, the initial equipment cost of at least \$250,000[2] would be necessary. In 1972 the Pennsylvania Department of Transportation spent \$36.00 per ton for gussasphalt in place. A conventional asphalt concrete at the same time was \$12.00 per ton, but the cost of gussasphalt was higher partly because of unfamiliarity with a new material and partly because of the additional cost of special equipment.[3]

REFERENCES

CHAPTER 3

GUSSASPHALT

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Table 3.1

Material Proportions of Gussasphalt Mixture[1]

Material	Percent of Batch Weight
Asphalt (Penetration 16)	7
Coarse aggregate	40.5
Fine aggregate	27.0
Filler	25.5
TOTAL	100.0

Table 3.2

Mineral Proportion by Sieve Size for Gussasphalt[1]

U. S. Standard Sieve No.	Percent of Batch Weight
Coarse Aggregate	
4	24.0
8	16.5
Fine Aggregate	
16	10.7
30	7.3
50	5.4
100	3.6
Filler	
200	9.0
-200	16.5

Table 3.3

Typical Mineral Component Gradation for Gussasphalt[1]

U. S. Standard Sieve No.	Percent Passing
3/8 in.	100
No. 4	74.2
No. 8	56.3
No. 16	44.8
No. 30	36.8
No. 50	31.0
No. 100	27.2
No. 200	17.7

Table 3.4

Effects of Different Factors on the Resistance of Asphalt and Gussasphalt Pavements to Wear by Studded Tires[6]

Material	Factors	Wear Ratio	Influence on Wear
Asphaltic concrete	Test temperature (14F versus 50F)	1:2 (1:0.6 to 3.3)	Very significant
	Preliminary treatment		
	Air versus 25 freeze-thaw changes		
	dense mixtures	--	Not significant
	open mixtures	1:1.2 to 2.2	Significant
	Dense and open mixtures		
	Air storage	--	Not significant
	25 freeze-thaw changes	1:1.2 to 1.8	Significant
	Quality of Aggregate $LD_{B/ASTM}$ 17:32	1:1.0 to 1.5	Significant
Gussasphalt	Gussasphalt versus Asphaltic concrete (AC)		
	Test temperature		
	50F	1:2	Significant
	32F	1:2.6	Significant
	14F	1:1	Not significant except for open AC after 25 freeze-thaw changes

(Continued)

Table 3.4 (Concluded)

Material	Factors	Wear Ratio	Influence on Wear
	Test temperature 50F:32F:14F	1.3:1:1.5 --	Significant
	Quality of aggregate	--	Not significant
	Preliminary treatment Air versus 25 freeze-thaw changes	--	Not significant

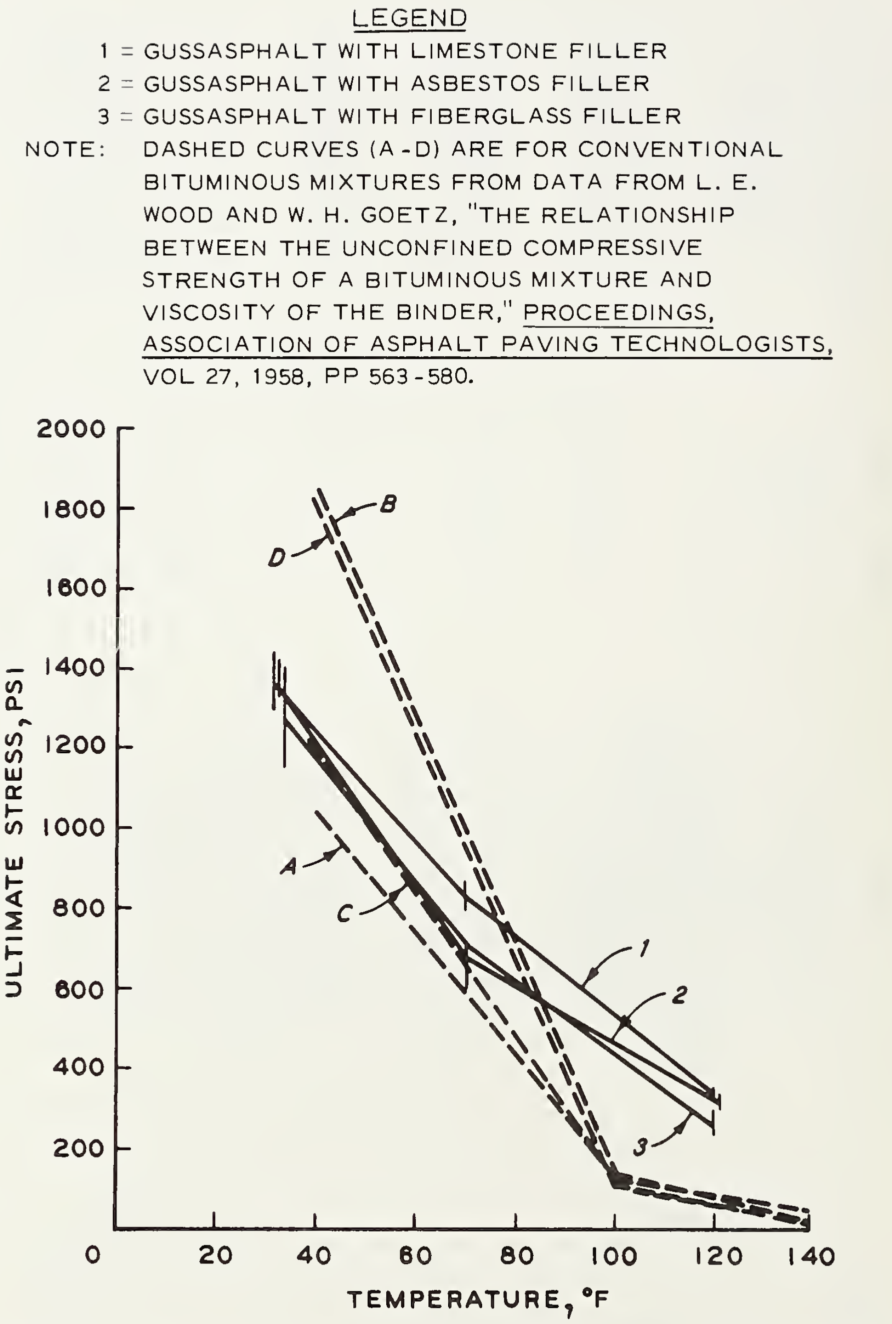

Figure 3.1. Effect of temperature on ultimate compressive strength of Gussasphalt[1]

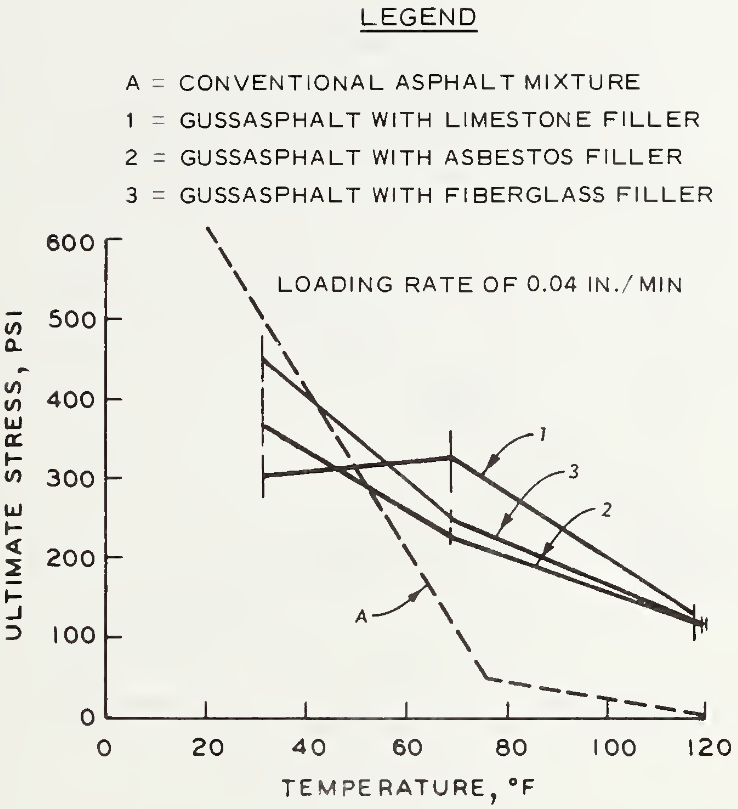

Figure 3.2. Effect of temperature on ultimate tensile strength of Gussasphalt (loading rate = 0.004 in./min)[1]

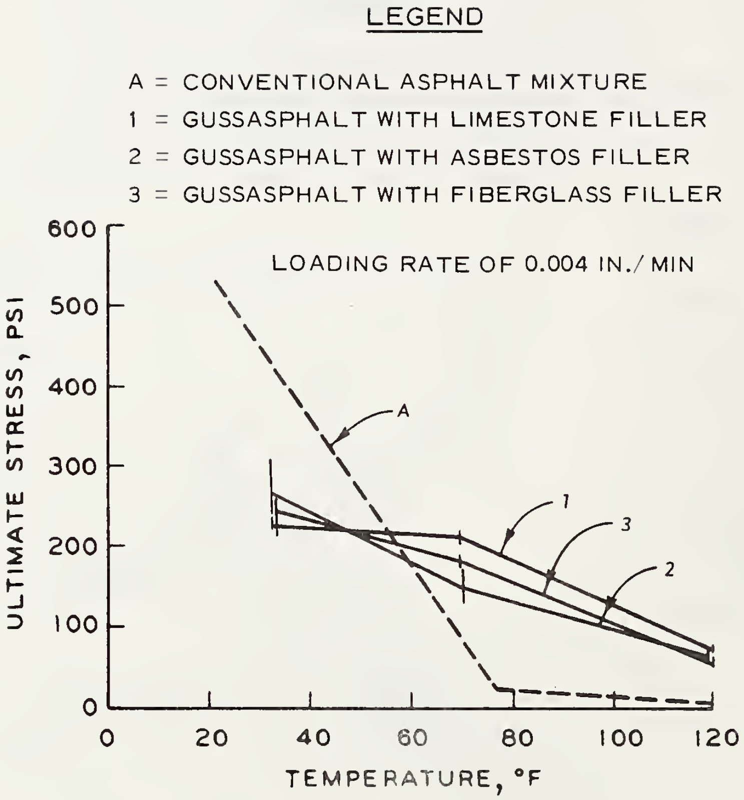

Figure 3.3. Effect of temperature on ultimate tensile strength of Gussasphalt (loading rate = 0.04 in./min)[1]

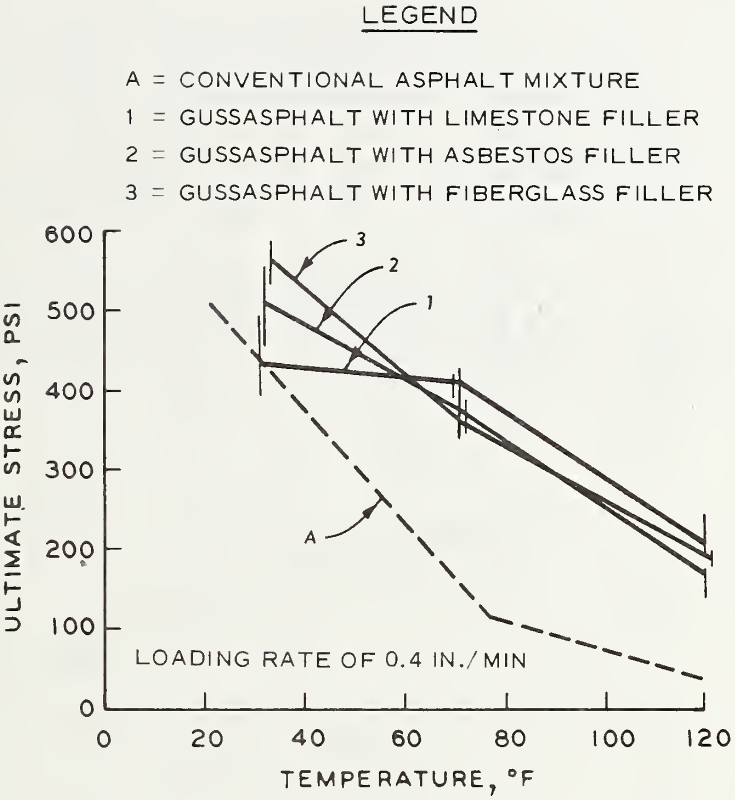

Figure 3.4. Effect of temperature on ultimate tensile strength of Gussasphalt (loading rate = 0.4 in./min)[1]

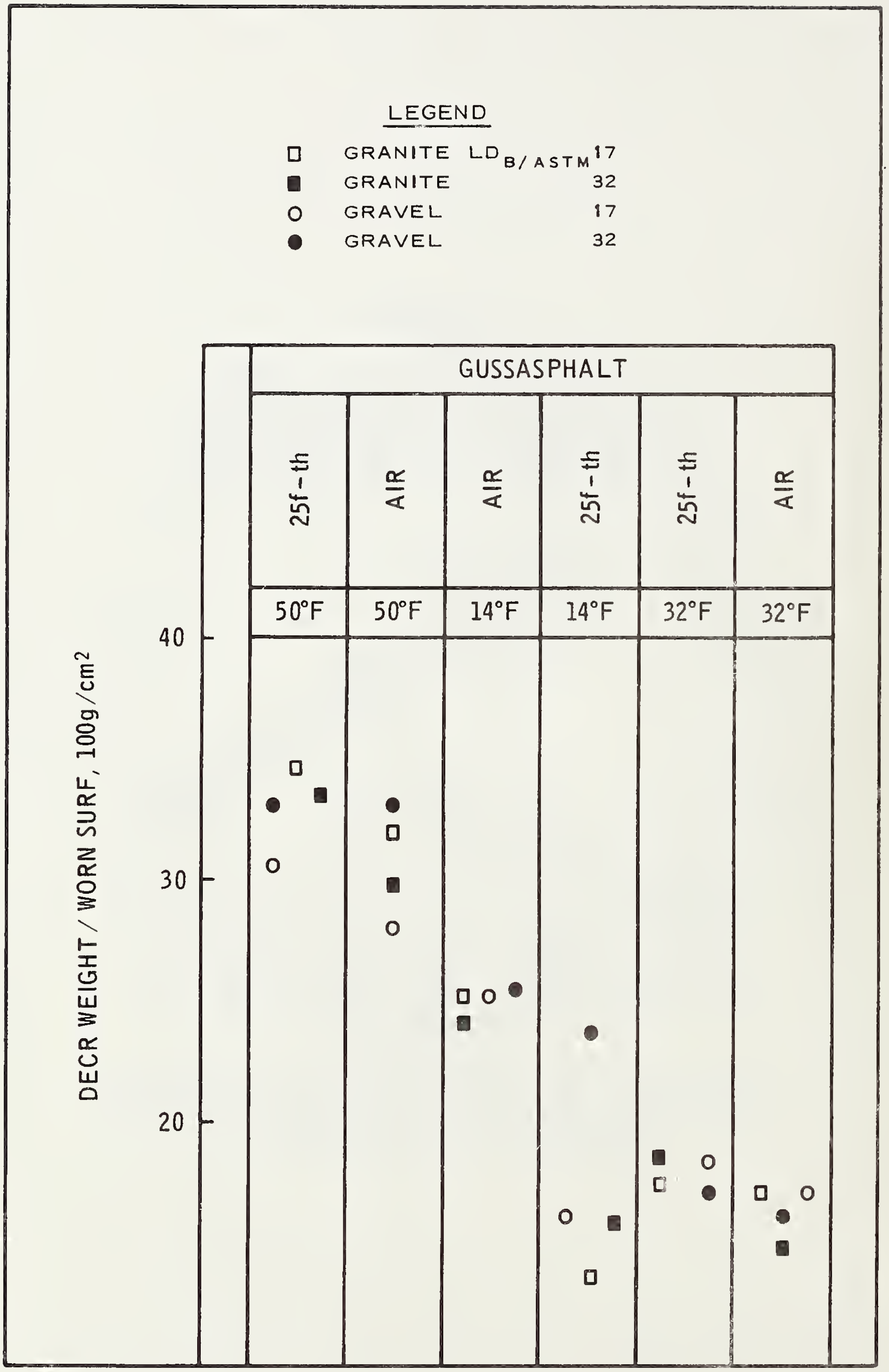

Figure 3.5. Results of wear of Gussasphalt[6]

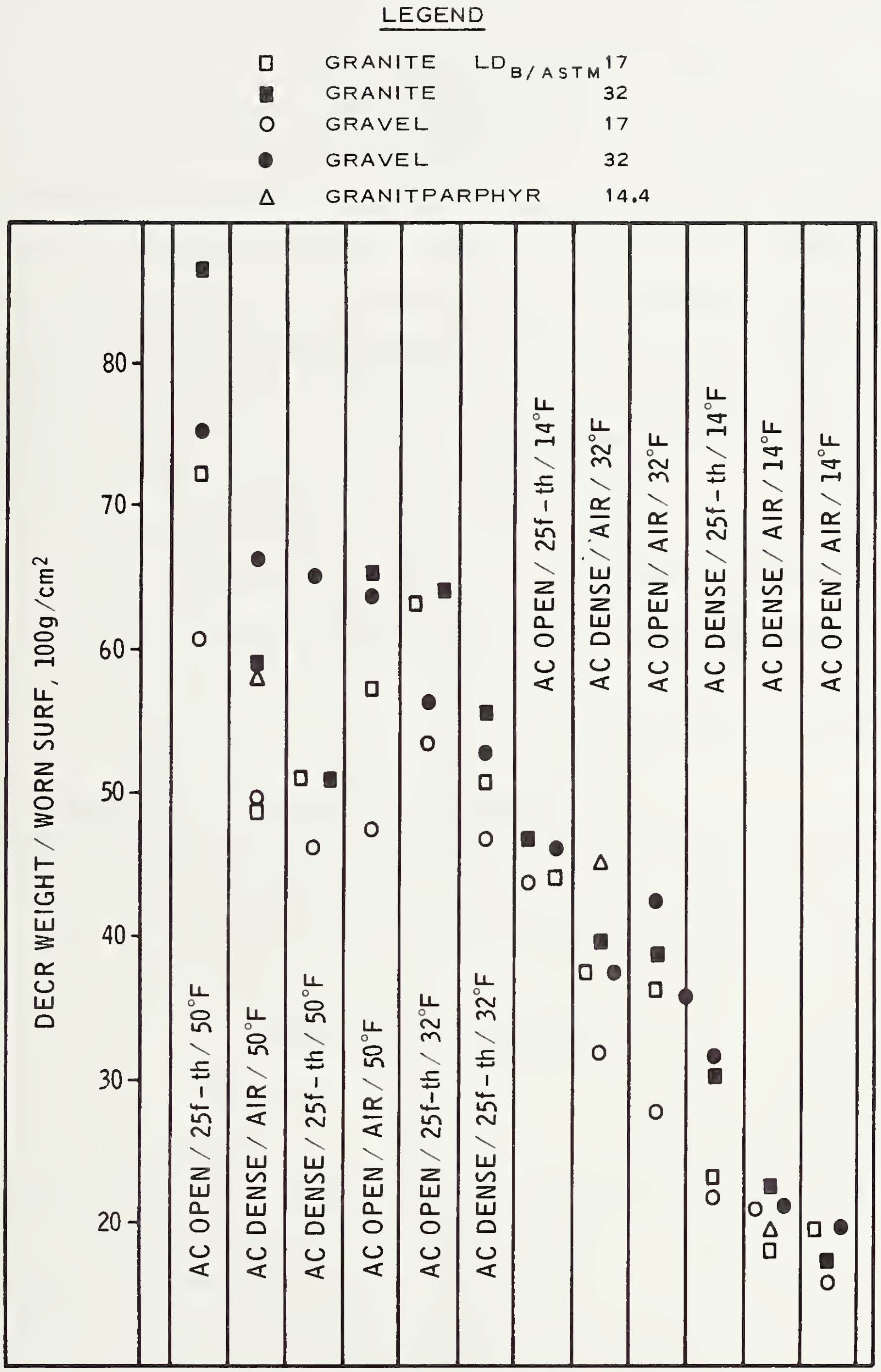

Figure 3.6. Results of wear of conventional asphalt concrete[6]

CHAPTER 4

ASBESTOS ASPHALTS

INTRODUCTION

4.1 BACKGROUND

The use of asbestos in pavement construction is not new. Asbestos has been used in bituminous joint filling compounds, seal-coating compounds, and asphalt bridge planking. In 1917 and 1918, patents were obtained by Warren Brothers Company of Boston for the use of asbestos in sheet asphalt to prevent bleeding of the asphalt during hot weather.[1] Although the use of asbestos in asphalt mixtures dates back many years, the amount of asbestos asphalt pavements constructed each year is small compared to the amount of conventional asphalt concrete pavement constructed each year.

4.2 DESCRIPTION OF MATERIAL

Asbestos is a hydrous magnesium silicate mineral ($Mg_6(OH)_3Si_4O_{10}$) with a fibrous structure.[1] There are several different grades or groups of asbestos fibers (see Table 4.1), but the asbestos that is generally used in pavement construction is a short fiber type, such as a 7M type.

MECHANICAL PROPERTIES

4.3 COMPRESSIVE STRENGTH

4.3.1 Effectsof Amount and Type of Binder. Asbestos asphalt mixtures react to changes in binder content the same way that a conventional asphalt mixture might react. Table 4.2 gives the average results of compressive tests on asbestos asphalt mixtures with varying asphalt contents. These unconfined compression tests were performed on specimens that were air cured and on specimens that were cured in water and indicate that the compressive strength of an asbestos asphalt mixture increases as the asphalt content is increased to an optimum value and then decreases with additional increases in asphalt content. Data regarding the effects of the type of binder were limited.

4.3.2 Effect of Aggregate, Filler, and Fiber. Some researchers[6] have presented data (Figure 4.1) that show a significant increase in the compressive strength of a mixture which has incorporated a small percentage of asbestos into the mixture. Yet others[1,7] present data (Tables 4.3 and 4.4) that show only a slight increase in compressive strength as a result of adding asbestos to the asphalt concrete mixture.

The size of the asbestos fiber has a small effect on the compression strength. Short fiber tends to give higher compressive strength values than do longer asbestos fibers (see Figure 4.1).

4.3.3 Effect of Placement Compactive Effort. The density of the asbestos asphalt concrete also affects the compressive strength. Specimens that were compacted to 96 to 97 percent of the laboratory density showed a 70 to 80 percent loss of compressive strength (see Table 4.2). Therefore, field compaction efforts would greatly affect the compressive strength of asbestos asphalt concrete mixtures.

4.3.4 Effects of Proportioning and Curing. Very little data showing the effects of different component proportions on the physical properties of an asbestos concrete mixture were available. Research[2] in the area of a ratio of asbestos fiber content to asphalt content indicates that a minimum ratio of 0.3 be used to prevent instability of the pavement under heavy traffic. An asbestos fiber content of 3 percent by total weight is recommended as the maximum asbestos content.[8]

The only data relating to the effects of curing on the compressive strength are presented in Figure 4.1 and Table 4.2. Laboratory prepared specimens[6] were compacted by applying a 3000-psi compressive load for 2 minutes to each standard test cylinder (ASTM D-1074 test method). Some specimens were cured for 4 days at room temperature and some were cured 4 days in a water bath at 120F. Unconfined compression tests (ASTM D-1074) on these specimens indicate that air cured specimens of asbestos asphalt concrete have a higher compressive strength.[6]

4.3.5 Effects of Temperature. Although the data on temperature and its effect on the compressive strength of asbestos asphalt concrete are limited, some conclusion might be drawn from the data of Table 4.5 The compressive strength of an asbestos asphalt concrete at 77F is 1003 psi, but the same mixture at 140F has a compressive strength of 318 psi. From this bit of information, it appears that an asbestos asphaltic concrete reacts to temperature changes in approximately the same fashion as a conventional asphaltic concrete. A conventional asphaltic concrete increases in compressive strength, bearing capacity, and stability as the temperature decreases.[9] The compressive strength at 77F may be as much as 4.6 times the strength at 140F.[9]

4.4 MARSHALL STABILITY

4.4.1 Effects of Amount and Type of Binder. As with compressive strength, the stability of the mixture increases with asphalt content until an optimum amount is reached and then with further asphalt contents it begins to decrease (see Table 4.6).[3] Several researchers[1,4,5,6] indicate that when asbestos is used in an asphaltic concrete the binder content can be increased by 1 to 3 percent by weight above the binder content of a conventional asphalt concrete without a reduction in the stability value.

The results of the stability test given in Table 4.3 indicate that a low penetration binder will produce a stronger mixture.

4.4.2 Effects of Aggregate, Filler, and Fiber Materials. Table 4.6 shows that the type of aggregate used affects the stability. In the case of the data shown, a crushed stone gives improved results over a crushed gravel. Increasing the asbestos content, in general, increases the marshall stability (Table 4.2).

4.5 FLEXURAL STRENGTH

Sufficient data to properly evaluate the effects of various factors such as binder type, aggregate, curing, temperature, etc., were not available. The data that were obtained (see Tables 4.7, 4.8, and 4.9) tend to indicate that the use of asbestos in an asphaltic concrete mixture does not significantly increase the flexural strength. In some cases, the flexural strength is decreased with the addition of asbestos fibers.

4.6 TENSILE STRENGTH

4.6.1 Effects of Amount and Type of Binder. Tensile testing by Tons and Krokosky[10] indicates that as the binder content of an asbestos asphaltic concrete is increased from 5.5 to 7.5 percent, the tensile strength also shows a slight increase. The effect that binder content has on an asbestos asphaltic concrete can be seen in Figures 4.2a and 4.2b. In the case of a bituminous concrete which used 5 percent asbestos fibers by dry weight of aggregate as the filler material, the tensile strength (Figure 4.2) increased by approximately 65 psi at 20F and by approximately 55 psi at 77F as the asphalt content was increased from 5.5 to 7.5 percent. Figure 4.2c represents a control mixture with 5 percent limestone filler.

4.6.2 Effects of Aggregate, Fillers, and Fibers. Asbestos fiber is usually of such a size that it can be considered a filler material for the bituminous concrete. Figure 4.3 represents tensile strength data collected by Tons and Krokosky[10] when the filler material was varied. Specimens containing either 5 percent limestone filler (LS), 5 percent asbestos fiber (A), or a 5 percent 50-50 mixture of asbestos and limestone (ALS, 2-1/2 percent asbestos plus 2-1/2 percent limestone) were tested in an Instron testing machine. The 5-in.-high by 6-in.-diam specimens were tested at -20F, 20F, 77F, and 120F. Mixtures with the asbestos-limestone filler yielded the highest tensile strength of approximately 500 psi at 20F. This was 100 to 150 psi greater than the control mixtures containing 5 percent limestone.

Tensile testing by Hickey[6] also indicates an increase in tensile strength of asphaltic concrete containing asbestos fibers (see Table 4.10). Cylindrical specimens (4 in. by 4 in.) were tested at a constant loading rate of 0.2 in. per minute. Some of the specimens were compacted at laboratory standard density and others were compacted to 96 to 97 percent of the laboratory standard density. The standard density specimens had a tensile strength of 230 psi for 7M fiber asbestos and 213 psi for the 6D fiber asbestos. The control specimens (no asbestos fibers) at standard density had a tensile strength of 143 psi. The addition of asbestos to the laboratory standard denstiy mixture provided an increase of approximately 60 percent for 7M asbestos fibers and 50 percent for 6D fibers. Tests on the 96 to 97 percent laboratory standard density did not show a significant increase in the tensile strength. The failure of the 96 to 97 percent density specimens to obtain a higher tensile strength might in part be due to a higher voids content. Tons and Krokosky[10] concluded that voids generally tend to reduce the tensile strength because higher void content decreases the cross sectional area and acts as inducers of stress concentrations.

Testing by Bishop[7] in which asbestos asphaltic concrete briquets were used did not show any significant increase in tensile strength. The test results[7] (Table 4.11) were inconsistent, and the effect of the asbestos fiber could not be determined.

4.6.3 Effects of Proportioning and Curing.

A minimum asbestos to asphalt ratio of 0.3[2] and a maximum asbestos content of 3 percent[8] are recommended to improve the physical properties of a bituminous concrete. Figure 4.3 shows that mixtures with 2-1/2 percent asbestos fiber (ALS) have significantly higher tensile strength than the mixtures containing 5 percent asbestos (A). The 5 percent asbestos mixture has a high void content (Figure 4.4) which may account in part for the lower tensile strength.

No information of the effects of curing on tensile strength could be found.

4.6.4 Effects of Temperature.

The effect of temperature on the tensile strength can be seen in Figures 4.2 and 4.3. As the temperature of a bituminous concrete decreases, the viscosity of the binder becomes greater, and the binder begins to act more like an elastic material. At high temperature the binder begins to act more fluid like and its tensile strength is greatly reduced. The tensile strength of a conventional asphalt concrete is approximately the same as an asbestos asphalt concrete at temperatures above 77F. In Figure 4.3c the tensile strength at 77F is approximately 100 psi for all the bituminous concrete mixtures tested. Only below 77F is there any significant difference between a conventional and an asbestos asphalt concrete. At 20F and below the asbestos mixtures have tensile strength of approximately 500 psi for the 2-1/2 percent asbestos mixture and 450 psi for the 5 percent asbestos mixtures. Conventional asphaltic concrete mixtures at 20F have an approximate tensile strength of 350 psi. At 120F the tensile strength of all the mixtures approaches zero.

4.7 DIMENSIONAL CHANGES

Although documentation on the dimensional stability of asbestos asphalts could not be located, it is generally believed that the material is relatively stable at normal ambient temperatures.

4.8 DURABILITY

4.8.1 Freeze and Thaw Resistance. In 1967, 28 miles of an asbestos asphalt concrete pavement were placed on Rogers Pass of the Trans-Canada Highway in Glacier National Park. The area has very high amounts of precipitation (30 to 50 in.) and snowfall (300 to 700 in.), and the weathering conditions are severe.[5] After six years of satisfactory performance, the asbestos asphalt pavement has not shown any visible wear, and the asphalt extracted from asbestos asphalt concrete field samples has retained a higher penetration value than the conventional asphalt samples. Table 4.12 presents the 1973 Glacier National Park penetration data. The higher penetration value (85) indicates that the binder in the asbestos asphalt mixture is not hardening as fast in the binder in the conventional asphalt mixture. This higher penetration asphalt would help contribute to less cold weather pavement cracking. A reduction in transverse cracking, when compared to nonasbestos sections, was apparent in the asbestos asphalt pavement through Rogers Pass.

4.8.2 Chemical Resistance. No information on the chemical resistance of asbestos asphalt could be located.

4.8.3 High Temperature Resistance. In a study by Speer and Kietzman,[11] asbestos modified mixtures were found to reduce the amount of rutting at high temperatures. Figure 4.5 shows the amount of rutting of a typical bituminous concrete when tested at high temperatures on a traffic simulator. The amount of rutting of the asbestos modified mixture was not greatly affected by the temperature until the temperature approached 130F, then the rate of rutting increased rapidly. Figures 4.6 and 4.7 show how the asbestos fiber to asphalt ratio and the asphalt penetration influence the amount of pavement rutting. At each fiber to asphalt ratio (f/a), there is a critical temperature above which the rate and amount of pavement rutting increases greatly. Lower penetration asphalts and higher f/a ratios increase the critical rutting temperature.

Figure 4.5 indicates that conventional asphalt concretes are highly dependent on the amount of traffic to produce rutting. The asbestos modified mixtures are more independent of the amount of traffic than the conventional mixtures, but once the critical temperature of the mixture is reached, the asbestos modified mixture reacts like a nonasbestos asphalt concrete.

4.9 ABRASION RESISTANCE

No information could be found on the abrasion resistance of asbestos asphalt.

4.10 TOUGHNESS

No information on the toughness of asbestos asphalt could be found although, as a general rule, the toughness of most composites is improved with the addition of fibers.

4.11 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

No information on the fatigue strength and dynamic response of asbestos asphalt could be found but, as in the case for toughness, in general these characteristics of composites are improved with the addition of fibers.

4.12 PERMEABILITY

The amount of permeability data available was limited. Only test results by Hickey[6] were obtained and Table 4.13 represents those test results. In the permeability tests, 3-1/2-in.-diam by 3-1/2-in.-high asbestos asphalt specimens were used. A water permeability test was conducted. All the standard laboratory density specimens were found to be impermeable, but at the 96 to 97 percent density, the permeability increased to 123.35 cu ft/sq ft/day with a head of 11.55 ft. The larger 6D fiber produced a permeability of over 400 cu ft/sq ft/day at a 11.55-ft head. Much of the impermeability was the result of the asbestos fiber's tendency to bulk.[6] This bulking increased the air void contents to near 5 percent in the test specimens.

The difference in the asbestos fibers resulted in a large difference in the water permeability. The 6D fibers showed a water permeability that was three to four times greater than the permeability when using the 7M fibers.[6] The finer (7M) asbestos fibers provided the lowest permeability of the mixtures studied.

USE OF ASBESTOS ASPHALT

4.13 EASE OF CONSTRUCTION

Many miles of asbestos asphaltic concrete pavement have been constructed, but information regarding the newest installations was difficult to obtain. Therefore, the information and data relating to current construction practices are limited.

Based on information of earlier asbestos asphalt construction, the asbestos asphalt concretes are placed using the same methods as conventional asphalt concrete. The asbestos modified mixtures will resist compaction efforts more than the conventional mixtures, but generally asbestos asphalts are constructed with the same ease as an asphaltic mixture.

In 1967, 28 miles of an asbestos asphalt pavement were placed in Canada. The pavement is part of the Trans-Canada Highway that passes through the Rogers Pass of Glacier National Park. The amount and type of traffic using this asbestos asphalt pavement was not available. The asbestos modified mixture was placed in 1-1/2- to 2-in. lifts. The performance as of 1974 has been "most satisfactory."[5]

In 1961, Bridge Street in Madawaska, Maine, was reconstructed from station 0+19 to station 8+55.75.[4] Two percent asbestos was used in the base, binder, and wearing course on the Bridge Street project. After seven years of service, a report[4] on the pavement performance concluded that the asbestos modified pavement had not cracked as extensively as the conventional pavements in the area, and the surface conditions appeared superior to the conventional pavements. The daily traffic using Bridge Street in 1960 was 3,685 vehicles. The ADT for 1980 was estimated to be 5,160.[12] Bridge Street carries some heavy truck traffic as a result of a paper mill which is located on both sides of the street, but the amount of heavy traffic was not stated.

Table 4.14 gives the location of some early asbestos asphalt pavement construction. A recent performance evaluation of these pavements was not located. Reference 2 gives a detailed evaluation 2-1/2 to 3 years after the construction of the pavements listed in Table 4.14, at which time the asbestos modified pavements were performing better than the conventional asphalt pavements.

In 1961, Shell Avenue in Contra Costa County, California, was overlaid with a 2.5 percent asbestos modified asphaltic concrete. The overlay was constructed over an existing pavement which exhibited large deflections under the large amount of truck traffic that used the Avenue. Sections were placed with and without asbestos fibers. A 3-year performance evaluation[13] concluded that the "addition of asbestos appears to offer no advantage for this project..." This conclusion was due in part to the amount of cracking in the asbestos modified sections. The nonasbestos sections exhibited less cracking than the asbestos modified sections.

4.14 EASE OF MAINTENANCE

Asbestos modified pavements should not be any more difficult to maintain than a conventional asphalt. Indications are that asbestos asphalt pavements are not maintenance free and will require periodic maintenance but perhaps at longer time intervals than conventional asphalt pavements.

4.15 ENVIRONMENTAL COMPATIBILITY

Asbestos asphalt in-place does not create any problems with the environment. Loose asbestos fibers would be dangerous to human health if they are suspended in the air, but asbestos can be incorporated into the asphalt mixture without being a health hazard if it is added into the pugmill while in unopened, low melt polyethylene bags.

4.16 AVAILABILITY AND COST

Most of the materials and equipment necessary to produce and construct an asbestos asphalt concrete pavement are generally available in the local area with the exception of the asbestos fibers. Asbestos fibers are usually obtained from mines in Canada, but sufficient quantity can be obtained without difficulty. The same equipment used to construct a conventional asphalt concrete is all that is required for an asbestos asphaltic concrete.

The use of asbestos fibers would increase the cost of pavement construction because more asphalt cement and additions of asbestos fibers would be required. The exact cost of placing asbestos asphalt concrete depends on the cost of local materials and the shipping cost.

At present, the F.O.B. price of asphalt cement is approximately $64/ton, and the F.O.B. Quebec price of asbestos is $75/ton. Using a minimum fiber to asphalt cement ratio of 0.3, the cost of asbestos in a mixture would be approximately one-third the cost of the asphalt cement in the mixture. As the price of asphalt cement continues to increase, the cost ratio of asbestos to asphalt will decrease, but the total cost will increase.

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CHAPTER 4

ASBESTOS ASPHALT

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Table 4.1

Asbestos Fiber Classification[2]

Quebec Standard Classification		Quebec Standard Test[a] Guaranteed Minimum Wt (oz)			
Group	Item	1/2 In.	No. 4	No. 10	Pan
3	3R	2	8	4	2
4	4K	0	4	9	3
5	5R	0	0	10	6
6	6D	0	0	7	9
7	7D	0	0	5	11
	7F	0	0	4	12
	7H	0	0	3	13
	7K	0	0	2	14
	7M	0	0	1	15
	7R	0	0	0	16

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[a] Canadian chrysotile asbestos classification.

Table 4.2

Average Compression Test Results of Asphalt Mixtures Containing Asbestos Fibers[6]

Asphalt[a] Content %	Cure Condition 4 days*	Avg. Compressive Strength, psi,** at 77°F	
		At Laboratory Standard Density	At 96.97 Percent of Laboratory Standard Density
		7M Fiber Asbestos	
8.0	Air	871.8	
8.0	Water	772.3	
8.5	Air	1002.8	294.0
8.5	Water	879.9	210.0
9.0	Air	769.1	
9.0	Water	712.5	
		6D Fiber Asbestos	
8.0	Air	831.8	
8.0	Water	707.8	
8.5	Air	968.3	215.0
8.5	Water	736.9	147.3
9.0	Air	860.9	
9.0	Water	644.3	

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* Air cured at room temperature; water cured at 120°F.

** Specimens tested at constant rate of deformation of 0.2 in. per minute.

a Penetration of asphalt cement was 50 to 60 units.

Table 4.3

Average Compression and Stability Properties of Asbestos Asphalt Mixtures[7]

				Marshall Stability[a]			
				85/100 Pen		40/50 Pen	
Percent Asbestos	Percent Natural*	Percent LSD**	Compression Strength psi†	Stability lb	Flow 0.01 in.	Stability lb	Flow 0.01 in.
0	2	0	290	1320	7	2280	8
0.5	2	0	321	1130	9	2190	12
1.0	2	0	415	930	13	2520	12
2.0	2	0	523	1670	5	3050	6
0	0	2	300	1360	8	2580	7
0.5	0	2	343	1460	8	2620	6
1.0	0	2	450	1570	7	2840	9
2.0	0	2	489	2190	16	3080	5
0	4	0	247	1650	8	2320	11
0.5	4	0	505	1270	17	2380	5
1.0	4	0	319	1380	7	2570	5
2.0	4	0	509	1970	8	3070	7
0	0	4	477	1890	10	2160	13
0.5	0	4	362	2330	9	3140	9
1.0	0	4	393	2320	7	2790	10
2.0	0	4	496	2610	8	3370	7
0	6	0	601	1680	7	2330	10
0.5	6	0	353	1980	6	3260	6
1.0	6	0	398	2100	7	2930	7
2.0	6	0	440	2470	8	3520	6
0	0	6	323	1530	7	2230	12
0.5	0	6	423	2020	8	2790	12
1.0	0	6	---	----	--	----	--
2.0	0	6	536	2600	8	2880	8

* A natural filler material found in the aggregate.

** A limestone dust filler.

† ASTM D 1074-60, 85/100 pen asphalt.

a 75-blow Marshall.

Table 4.4

Compression Test Results on Specimen's Place on the Scajacada Creek Expressway in Buffalo, New York[1]

Mix	Asbestos Cont., %	Asphalt Cont., %	Density, gm/cc	Void Content, % vol.	Static Conc. Comp. Test			Comp. Strength,** psi	Marshall Values	
					Max. Load, psi	Total Strength, psi-hr	Pen. at Fail. in.		Stability lb	Flow, 0.01 in.
2A	0	7.6	2.28	3.9	127	172	0.05	615	886	22
Mod. 2A:										
A	1.3	7.8	2.29	3.0	127	141	0.09	775	1245	16
B	2.5	8.4	2.28	2.2	510	922	--	872	1098	34
C	1.3	8.6	2.33	0	510	861	0.47	664	673	53

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* Specimens compacted by method described in ASTM D 1074-58T.
** ASTM method at 75 F.

Table 4.5

Asbestos Asphalt Concrete Compression Strength[6]

Mix	Asphalt Content %	Compression Strength, psi At 77 F*	Compression Strength, psi At 140 F**
Control	8.5	328.9	108.3
7M Asbestos	8.5	1002.8	318.5
6D Asbestos	8.5	968.3	346.0
96-97 Percent Laboratory Standard Density			
Control	8.5	--	--
7M Asbestos	8.5	294.0	78.6
6D Asbestos	8.5	215.0	80.0

* Specimens cured at room temperature for 4 days.
** Specimens cured at room temperature for 1 day.

Table 4.6

Comparison of Marshall Test Data for Asphalt Concrete and Asbestos Asphalt[3]

Without Asbestos Fibers					With Asbestos Fibers				
Asphalt Content, % by Wt	Bulk Density, pcf	Stability, lb	Flow, 0.01 in.	Voids, %	Asphalt Content, % by Wt	Bulk Density, pcf	Stability, lb	Flow, 0.01 in.	Voids, %
(a) Crushed Gravel Surface Course Mixture									
4.75	148.6	1,153	8	4.9	4.75	148.5	1,291	7	5.6
5.5	150.0	1,235	8	3.0	5.5	150.1	1,550	8	3.6
6.25	149.9	1,375	10	2.0	6.25	150.1	1,491	8	2.6
7.0	149.2	1,122	11	1.4	7.0	149.2	1,301	14	2.2
7.75	147.9	858	14	1.2	7.75	148.0	1,081	18	2.0
(b) Crushed Stone Surface Course Mixture									
4.75	152.1	2,256	9	5.1	4.75	148.9	1,809	8	7.1
5.5	152.6	2,109	9	3.9	5.5	151.0	2,028	11	4.5
6.25	152.2	1,655	12	3.3	6.25	151.4	1,841	12	3.1
7.00	151.4	1,248	14	3.0	7.0	150.8	1,462	15	2.2
7.75	149.7	889	21	3.2	7.75	149.4	1,195	19	1.8
(c) Crushed Stone Binder Course Mixture									
4.0	152.5	2,136	8	6.1	4.0	149.4	2,086	12	8.6
4.75	153.5	2,085	10	4.1	4.75	141.3	2,024	10	6.2
5.5	153.7	1,768	11	2.7	5.5	151.8	1,864	12	4.7
6.25	152.7	1,175	16	2.1	6.25	151.6	1,479	16	3.5
7.00	151.2	929	23	1.7	7.00	150.7	1,140	22	2.8
					7.75	149.4	954	33	2.2

Table 4.7

Flexural Strength of Asbestos Asphalt Concrete[7]

% Asbestos	% Natural*	% LSD**	Flexural Strength psi***
0	2	0	90
0.5	2	0	93
1.0	2	0	96
2.0	2	0	131
0	0	2	84
0.5	0	2	117
1.0	0	2	280
2.0	0	2	189
0	4	0	88
0.5	4	0	140
1.0	4	0	85
2.0	4	0	136
0	0	4	130
0.5	0	4	140
1.0	0	4	121
2.0	0	4	79
0	6	0	206
0.5	6	0	97
1.0	6	0	91
2.0	6	0	109
0	0	6	97
0.5	0	6	172
1.0	0	6	-
2.0	0	6	175

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* A natural filler material found in the aggregate.

** A limestone dust filler.

*** Beams specimens 1-1/2 by 1-1/2 by 8 in. long, 95/100 pen AC, loaded at 0.05 in. per minute.

Table 4.8

Asphaltic Concrete and Asbestos Asphalt Investigations

Flexural Strength Test Results[6]

Mix	Percent Lab Std Density	Density, pcf	Air Voids, %	Test Temp, °F	Loading Rate, in./min	Deflection at Max Load, in.	Modulus of Rupture, psi
Control							
8 percent asphalt	100	144.7	2.16	60	0.1	0.140	299.8
	100	144.6	2.22	60	0.1	0.132	314.6
	95	139.2	5.88	60	0.1	0.102	277.4
	95	139.6	5.61	60	0.1	0.202	324.9
7M fiber asbestos							
8.5 percent asphalt	100	142.8	1.76	40	0.5	0.019	547.8
	100	142.8	1.80	60	0.1	0.053	258.6
	95	135.9	6.53	60	0.1	0.037	190.6
	95	135.7	6.70	60	0.1	0.048	205.1
6D fiber asbestos							
8.5 percent asphalt	100	144.2	0.83	60	0.1	0.058	184.7
	100	143.7	1.17	60	0.1	0.064	208.0
	95	138.5	4.75	60	0.1	0.078	244.8
	95	138.6	4.68	60	0.1	0.068	233.0

Table 4.9

Flexural Strength Comparisons for Asbestos Asphalts and Asphaltic Concrete Showing Effects of Aggregates[3]

		Without Asbestos Fibers					With Asbestos Fibers				
Test Temp, °F	Asphalt Content, % by Wt	Bulk Density, pcf	Voids, %	Maximum Center Load, lb	Stiff-ness, lb/in.	Modulus of Rupture, psi	Bulk Density, pcf	Voids, %	Maximum Center Load lb	Stiff-ness lb/in.	Modulus of Rupture, psi
				(a) Crushed Gravel Surface Course Mixture							
40	5.7	146.1	5.3	1,100	3,280	1,030	145.0	6.5	1,280	3,600	1,200
	7.7	146.5	2.2	1,320	2,120	1,240	147.5	2.4	1,490	3,120	1,400
100	5.7	146.3	5.2	4	10	4	145.6	6.2	23	76	22
	7.7	145.3	3.0	14	18	13	147.5	2.4	34	40	32
				(b) Crushed Stone Surface Course Mixture							
40	5.5	148.8	6.3	1,400	4,000	1,000	144.1	8.0	880	4,000	820
	7.5	149.0	4.0	1,250	2,600	1,170	148.9	2.6	1,620	3,400	1,520
100	5.5	148.5	6.5	21	96	20	144.5	8.7	22	120	21
	7.5	149.3	3.8	20	20	19	149.5	2.2	36	88	34
				(c) Crushed Stone Binder Course Mixture							
40	5.2	149.9	5.6	1,280	4,400	1,200	146.2	8.7	1,000	4,960	940
	7.2	150.1	2.1	1,040	2,200	970	150.0	2.8	1,350	4,960	1,270
100	5.2	150.6	5.2	20	48	19	146.9	8.2	28	104	26
	7.2	150.5	1.8	14	20	13	150.1	2.8	34	80	32

Table 4.10

Asbestos Asphalt Concrete Tension Test Results[6]

Mix	Cylinder No.	Density, pcf	Air Voids, %	Elongation Before Failure, in.	Tensile Strength, %
		Laboratory Standard Density			
Control	9	145.8	1.77	0.06	142.9
8.0 percent asphalt	34	146.6	1.25	0.07	146.9
	32	145.7	1.87	0.06	138.5
	Average	146.0	1.63	0.07	142.8
7M fiber asbestos	21	145.9	0.02	0.06	229.8
8.5 percent asphalt	22	146.2	0.00	0.07	234.0
	45	145.8	0.06	0.06	225.2
	Average	146.0	0.03	0.07	229.7
6D fiber asbestos	25	146.1	0.00	0.07	226.8
8.5 percent asphalt	26	145.7	0.19	0.05	208.7
	46	145.5	0.28	0.04	203.0
	Average	145.8	0.16	0.05	212.8
		96-97 Percent Laboratory Standard Density			
Control	12	139.4	6.09	0.07	83.6
8.0 percent asphalt	39	141.2	4.90	0.07	94.7
	40	141.6	4.61	0.07	95.5
	Average	140.7	5.20	0.07	91.3
7M fiber asbestos	23	139.6	4.34	0.04	107.2
8.5 percent asphalt	24	139.5	4.38	0.05	119.1
	Average	139.6	4.36	0.05	113.2
6D fiber asbestos	27	138.7	4.94	0.04	116.2
8.5 percent asphalt	28	139.3	4.54	0.05	124.6
	47	138.5	5.07	0.04	113.9
	Average	138.8	4.85	0.05	118.2

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* Air cured 4 days at room temperature. Specimens tested at constant rate of loading of 0.2 in. per minute.

Table 4.11

Tensile Strength of Asbestos Asphalt Concrete[7]

Filler			Tensile Strength, psi
% Asbestos	% Natural*	% LSD**	
0	2	0	89
0.5	2	0	57
1.0	2	0	85
2.0	2	0	71
0	0	2	71
0.5	0	2	88
1.0	0	2	133
2.0	0	2	140
0	4	0	53
0.5	4	0	104
1.0	4	0	34
2.0	4	0	116
0	0	4	110
0.5	0	4	89
1.0	0	4	89
2.0	0	4	44
0	6	0	47
0.5	6	0	53
1.0	6	0	71
2.0	6	0	75
0	0	6	33
0.5	0	6	122
1.0	0	6	-
2.0	0	6	71

* Natural filler material in the aggregate.
** Limestone dust filler.

Table 4.12

Glacier National Park Penetration Results, 1973[5]

Standard Mix	- Asphalt content	= 5.7%
	Penetration	= 58 (43% of original)
1% Asbestos modified	- Asphalt content	= 6.5%
	Penetration	= 63 (47% of original)
2% Asbestos modified	- Asphalt content	= 7.5%
	Penetration	- 85 (60% of original)

Table 4.13

Asbestos Asphalt Concrete Permeability Test Results[6]

Mix	Asphalt Content, %	Density, pcf	Voids, %	Permeability, cu ft/sq ft/day		
				11.55 ft Water	23.10 ft Water	34.65 ft Water
	Laboratory Standard Density					
Control	8.0	147.9	0.36	0.00	0.00	0.00
	8.0	148.5	0.00	0.00	0.00	0.00
7M fiber asbestos	8.5	145.9	0.02	0.00	0.00	0.00
	8.5	146.1	0.00	0.00	0.00	0.00
6D fiber asbestos	8.5	144.3	1.12	0.00	0.00	0.00
	8.5	144.4	1.05	0.00	0.00	0.00
	96-97 Percent Laboratory Standard Density					
Control	8.0	143.6	3.25	0.00	0.00	0.00
	8.0	143.6	3.25	0.00	0.00	0.00
7M fiber asbestos	8.5	139.2	4.61	123.35	270.06	418.86
	8.5	139.6	4.34			
6D fiber asbestos	8.5	138.3	5.23	443.20	912.87	1,233.68
	8.5	138.3	5.23	518.71	815.12	1,201.20

Table 4.14

Asbestos Asphalt Concrete Pavements[2]

Location		Date Placed	Placed by	Estimated Traffic	
City	Street			Count per Day	% Heavy Trucks
Calgary, Alberta	Alyth Freeway	June 1960	City Street Dept.	6,500	10 (est)
Dallas, Texas	Greenville	Nov. 1959	M. P. McInerney Co.	23,000	
	Ross Avenue			17,000	15 (est)
Manville, New Jersey	N. 13th	Sept. 1959	Jannarone Engineering Company	500 (est)	10 (est)
St. Louis, Missouri	Manchester	Oct. 1959	Bridges Paving Co.	10,300	25
	(Tower Grove) Strodtman			500-1,000 (est)	min. 10 (est)

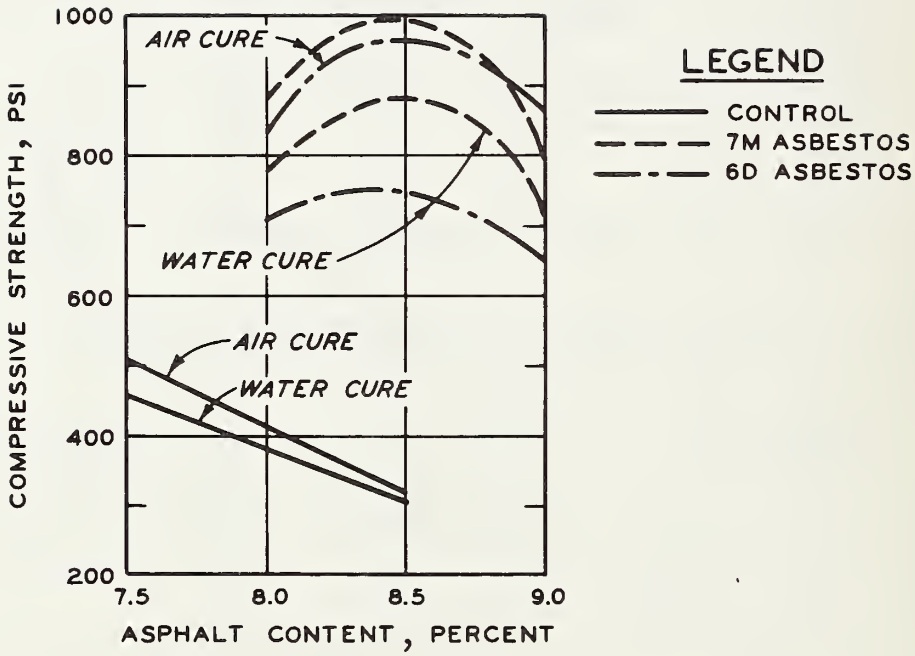


Figure 4.1. Compressive strength of asbestos asphalt concrete mixtures at laboratory standard density[6]

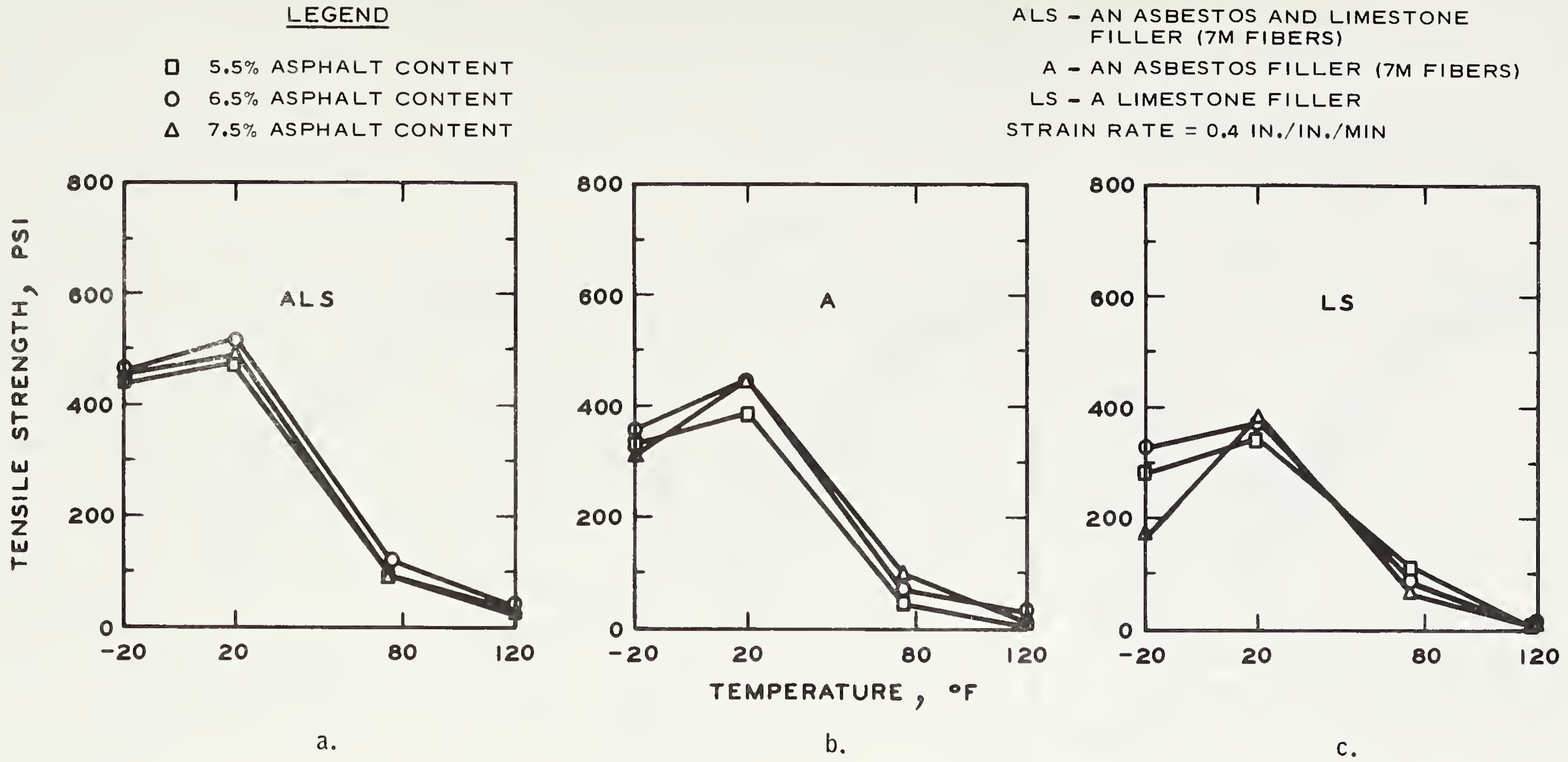
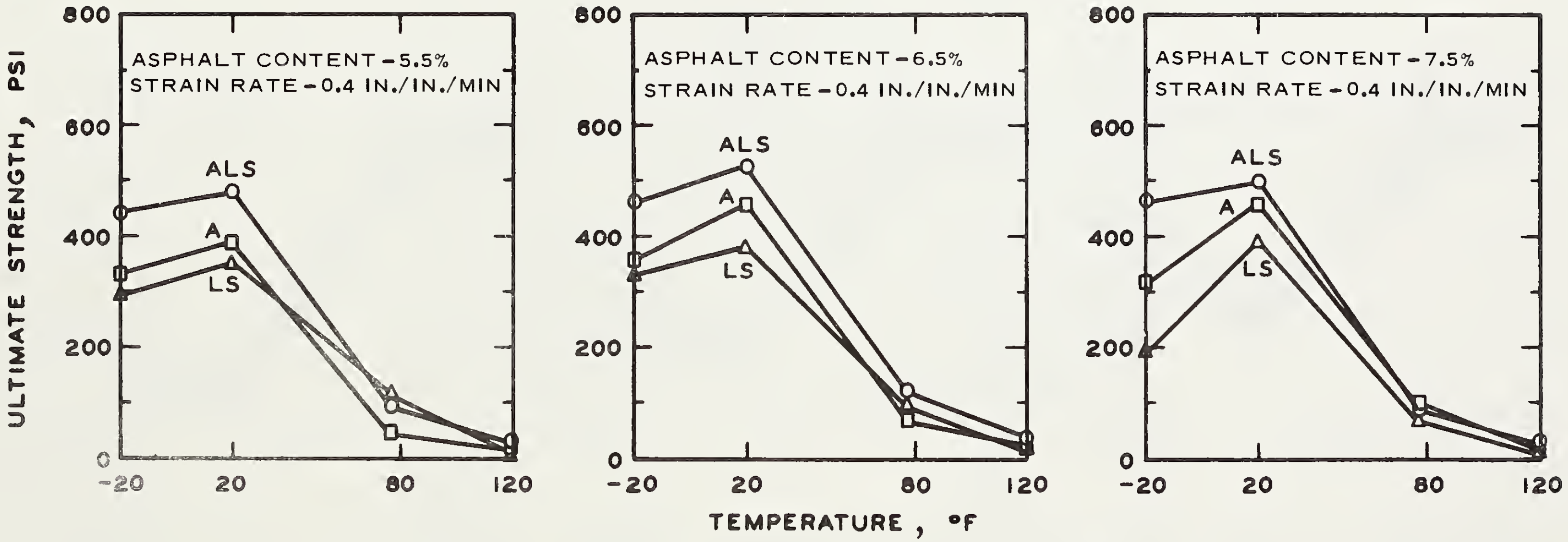


Figure 4.2. Tensile strength of asbestos asphalt as a function of temperature and asphalt content[10]



a. ASPHALT CONTENT = 5.5%

b. ASPHALT CONTENT = 6.5%

c. ASPHALT CONTENT = 7.5%

Figure 4.3. Tensile strength of asbestos asphalt as a function of temperature, filler, and asphalt content[10]

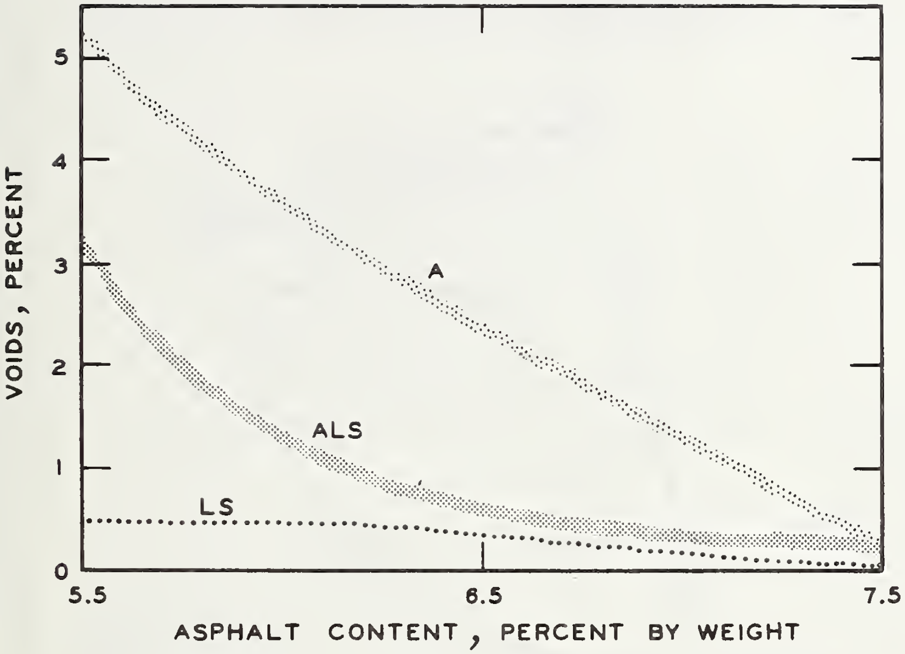


Figure 4.4. Void content in mixtures of various asphalt contents[10]

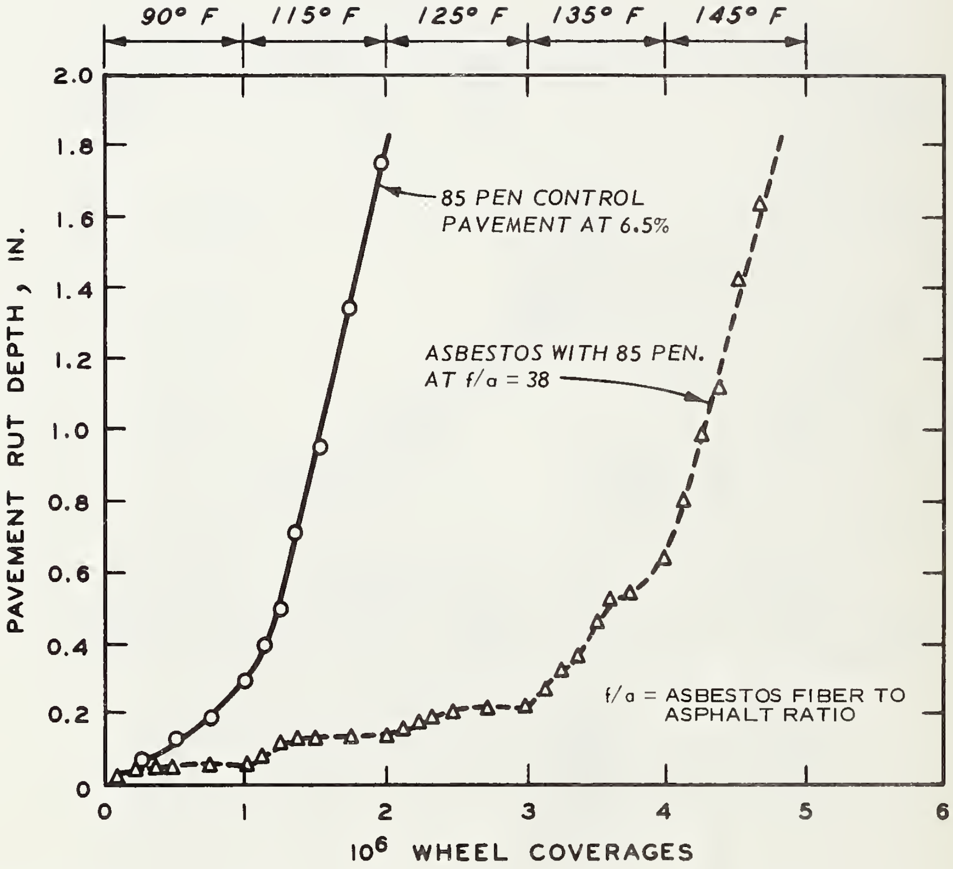


Figure 4.5. Typical performance of asbestos asphalt under wheel loadings at elevated temperatures[11]

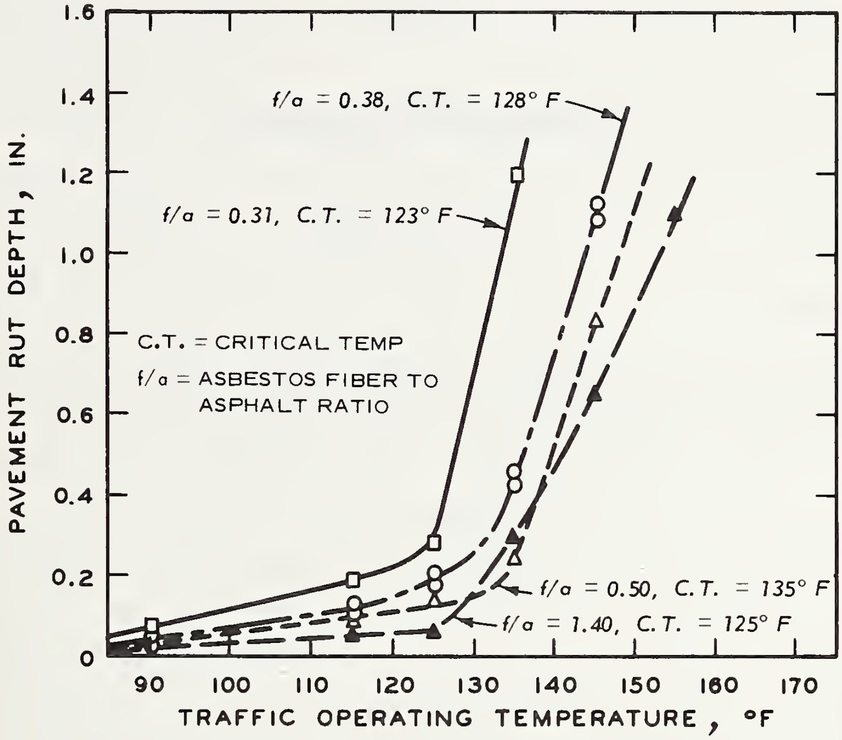


Figure 4.6. Performance of asbestos pavements with 85 penetration asphalt[11]

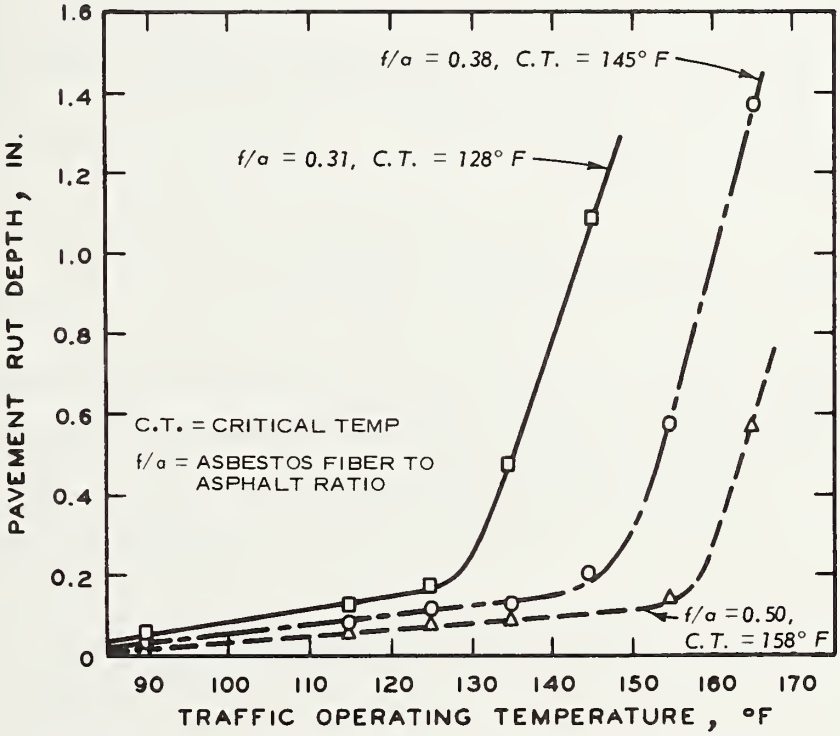


Figure 4.7. Performance of asbestos pavements with 60 penetration asphalt[11]

CHAPTER 5

SULFUR MODIFIED ASPHALTS

INTRODUCTION

5.1 BACKGROUND

In the early 1900's before air blowing became the major mode of modifying asphalt characteristics, sulfur was used to modify the asphalt properties. Several patents were issued for sulfur modified pavement mixtures, but only marginal improvements were made in the mistures, and eventually the interest in sulfur modified asphalt pavement fell by the wayside.

In the last few years, interest in sulfur modified asphalts has been renewed because of an over supply of sulfur and shortages in petroleum products and quality aggregates. It has been estimated by the Federal Power Commission[1] that by the year 2000 the supply of sulfur will reach 43,000,000 tons. Much of the recent research into the use of sulfur modified asphalts has been conducted in Canada, where in 1964 full-scale test sections were placed.[2]

5.2 DESCRIPTION OF MATERIAL

Elemental sulfur is a yellow crystalline solid with a relative density of 2.0. Between the temperatures of 234F and 300F sulfur is a liquid with a viscosity of 8 to 10 poises. Above 315F, the viscosity of sulfur begins to increase rapidly and becomes too high to be mixed with asphalts (Figure 5.1). At the processing temperatures of asphalt, sulfur is a liquid and can be combined in the asphalt concrete. After cooling, the sulfur solidifies in the voids of the asphalt concrete mixtures and acts as a filler.

Sulfur can be incorporated into the asphaltic mixture by either of the following two methods:

a. The liquid sulfur can be dispersed in the hot asphalt cement in any proportion desired, and then the sulfur-asphalt binder can be mixed with the aggregare.[4] The sulfur-asphalt binder is similar to an asphaltic emulsion, but the sulfur-asphalt (SA) is not very stable. After two to three hours, the sulfur will begin to settle out of the SA binder. Therefore, the SA binder must be mixed with the aggregate before separation of the sulfur and asphalt occurs. In this method, which was developed by Gulf Oil Canada Limited,[4] the sulfur is used to replace a portion (usually one-half) of the asphalt binder.

b. The other method, developed by Shell Canada Limited,[2,3] uses sulfur as an undiluted addition to the aggregate and asphalt combination. Liquid sulfur is added as a percentage of the dry aggregate weight. The amount of sulfur controls the workability of the sulfur modified asphalt concrete. If enough sulfur is added, the mixture can be poured into place. When using this method of adding sulfur, 10 to 15 percent liquid sulfur is generally required to make a workable mix. Mixtures formed by this method do not require compaction. Deme[3] stated that "...initial experimental efforts in constructing pavements revealed that conventional rolling resulted in damage to the sulfur crystal structure which formed as the mix cooled."

MECHANICAL PROPERTIES

Various physical properties of sulfur modified asphalt mixtures are discussed in the following sections. Only limited data were obtained, because research of sulfur modified asphalt is in its early stages.

5.3 COMPRESSIVE STRENGTH

Direct compressive strength information on sulfur modified asphalts is limited. Marshall stability values are available, however, and are presented in Section 5.4.

5.4 MARSHALL STABILITY

5.4.1 Effects of Amount and Type of Binder. In both the Gulf[4] and Shell[3] methods, the stability of the mixture is increased when sulfur is added to an asphaltic concrete mixture. Table 5.1 and Figure 5.2 represent data obtained when using the Shell[3] method. Table 5.1 gives the marshall results of increasing the sulfur content in an asphaltic mixture that contains 6 percent asphalt cement. The specimens were compacted with a two-hammer blow, one side only effort. As the sulfur content is increased from 8 percent to 16 percent, the marshall stability increases from 946 lb to 3105 lb; a substantial increase in stability for a small increase in sulfur. Figure 5.2 shows that as the asphalt content of a sulfur modified mixture is increased, an optimum stability value is reached, and any additional increase in asphalt content will only cause a loss in stability and an increase in flow. At 4 percent asphalt, the stability value is approximately 2800 lb with a flow of 0.06 in., but when the asphalt content is increased to 6.5 percent, the stability drops to 1265 lb and flow increased to 0.14. These specimens were not prepared by hammer compaction. Instead, the specimens were rodded 25 times with a spatula to obtain a bulk density of approximately 120 lb/ft^3. Increasing the asphalt content in a sulfur modified mixture has the same effect that is expected in a conventional asphalt mixture, i.e., there is an optimum asphalt content. But increasing the sulfur content continues to increase the stability without substantial increases in flow.

The Gulf[4] method, which replaced part of the asphalt binder with sulfur to form a SA binder, yields marshall stability results that are similar to the results obtained by Shell[2,3] researchers. The marshall results obtained by Gulf[4] are listed in Table 5.2 and Figure 5.3. The specimens were prepared with a binder that contained 50 percent elemental sulfur and 50 percent by weight 85 to 100 penetration grade asphalt. Standard marshall test method ASTM D-1559 was used. Conventional asphaltic concrete mixtures were also tested and compared to the SA mixtures. The SA mixtures gave marshall stability values two to four times greater than the conventional asphalt mixtures. Both the conventional and SA mixtures showed an optimum binder content that would yield the highest stability value. Additional increases in the SA binder content decreased the stability values, but the rate of decrease was not as rapid as the decrease that occurs in a conventional asphalt mixture.

In all the stability data presented, the stability values increased when the amount of sulfur in the mixture was increased. Hammond and Deme[2] indicate that it may be possible to obtain a desired stability value by varying the sulfur content in the mixture.

5.4.2 Effects of Aggregates and Fillers. Many of the sulfur modified asphalt test specimens were prepared from poor quality aggregates. These aggregates, when used with an asphalt binder, would not meet construction specifications, but when sufficient sulfur is added, the same poor quality aggregate mixtures obtain high stability values, some above 2,000 lb. Stability values of 1,800 lb or higher are desired for most pavements with heavy traffic. Figure 5.4 shows stability of different aggregates as a function of sulfur content. Table 5.3 lists the gradations of the aggregates given in Figure 5.4 The sand aggregates obtained higher stability values than the gravel aggregates, but the reason for higher strength is not yet known. Some researchers[1] suggest that the aggregate's surface angularity may influence mixture stabilities. Aggregates used in conventional pavement usually have a high degree of surface angularity, and this angularity creates stress concentrations in the crystalline sulfur structure, thereby reducing the load-carrying capability. Some sands have surfaces that are more rounded and, therefore, may not contribute to stress concentrations. At present, the surface angularity theory has not been investigated.

5.4.3 Effects of Proportioning and Curing. The sulfur content should be at least equal to the asphalt content to improve the strength properties of the mixture. Research by Gulf[4] indicates that a 40:60 or 50:50 blend of sulfur to asphalt cement is a blend that will improve a bituminous mixture. Research[1] using the Shell mixture, "Thermopave," indicates a high sulfur content. A mixture with an 80.5 percent sand to 6 percent asphalt to 13.5 percent sulfur ratio was found to produce the desired workability and stability.[1]

No information on the effect of various types of curing on the compressive strength of sulfur modified asphalts could be found.

5.4.4 Effects of Temperature. No information on the effects of varying temperatures on the compressive strength of sulfur modified asphalts could be found.

5.5 FLEXURAL STRENGTH

The flexural strength data was very limited. Table 5.4 represents the only flexural strength data obtained. The flexural test specimens, 25/32 by 1-3/16 by 9-7/16 in. were prepared with 82 percent 150/200 pen asphalt and 12 percent sulfur. Two temperatures (50F and 68F) and two loading rates (0.008 and 0.787 in./min) were used. The highest flexural strength was 415 psi at 50F and 0.787 in./min. At 68F and 0.008 in./min, the lowest flexural strength was 46 psi.

5.6 TENSILE STRENGTH

Tensile test by Deme[3] indicates that sulfur modified asphalt concretes have a tensile strength of less than 100 psi when tested at 68F. Tests at 50F result in a two to three times increase in tensile stress. The results of Deme's[3] direct tensile test are presented in Table 5.5. The test specimens were prepared with a medium-coarse sand, 6 percent of a 150/200 penetration grade asphalt, and 12 percent sulfur. The mixture was formed into 1- by 1- by 4-in. size test specimens. Tensile test results reported by Saylak, Gallaway, Epps, and Ahmad[1] are presented in Tables 5.6 and 5.7. Direct tensile tests were run on 1-1/2- by 1-1/2- by 6-in. specimens. All the tensile tests were run in an Instron Universal Tensile Tester at 20F, 73F, and 135F.

5.6.1 Effects of Amount and Type of Binder. From the data given in Tables 5.6a and 5.6b, the tensile failure stress increases as the sulfur content is increased and the asphalt content decreased. The failure strain was decreased as the sulfur content was increased and the asphalt content decreased.

5.6.2 Effect of Aggregates and Fillers. Sand I from Table 5.6 is a poorly graded beach sand. It shows higher strength values than Sand II, which is a dense graded concrete sand. As previously discussed, the aggregate surface angularity may contribute to stress concentration which results in lower failure stresses.

5.6.3 Effects of Proportioning and Curing. No additional detailed information on either the effects of porportioning or curing on the tensile strength of sulfur modified asphalt other than that reported in the preceding paragraphs could be located.

5.6.4 Effects of Temperature. Most mixtures that contain asphalt will lose strength as the temperature is increased; sulfur modified asphaltic mixtures are not exceptions.

When compared to a conventional asphaltic concrete, the sulfur modified mixture generally has a smaller tensile failure stress and stiffness value, but at high temperatures (133F) the sulfur modified mix yielded a stiffness two to three times that of a conventional asphalt. This may indicate a lesser degree of viscoelasticity in the sulfur mixtures.

5.7 DIMENSIONAL CHANGES

No information on the dimensional stability of sulfur modified asphalts could be found.

5.8 DURABILITY

Data regarding the effects of freeze and thaw cycles on sulfur modified asphalts were not available, but several test sections have been placed in Canada for observation. These test sections are subject to severe weather and an evaluation at a later date will help indicate the amount of durability that can be expected.

No information on chemical, high temperature, or fire resistance could be found, but the presence of sulfur within melting temperatures being from 242F to 300F would probably adversely affect the performance of the modified asphalt when subjected to high temperatures and fire.

5.9 ABRASION RESISTANCE

The amount of abrasion on the test sections in Canada has not been evaluated. Only skid resistance measurements were made on one test section. These measurements (Table 5.8) were made after nine months of traffic. Deme and Hammond[2] also noted that the sulfur asphalt pavement had not shown any sign of being polished by traffic. Most of the sulfur asphalt pavements are fairly new and have not been used enough to fully evaluate the abrasion resistance.

5.10 TOUGHNESS

No information on the toughness of sulfur modified asphalt could be located.

5.11 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

Fatigue testing[2] of one sulfur modified asphalt mixture (sand aggregate) indicated that the fatigue life of the sulfur modified asphalt mixture was lower than the fatigue life of an asphalt concrete but slightly above the minimum fatigue life used by Shell.[2] The fatigue tests were conducted in a three-point bending apparatus at a constant frequency of 50 Hz and temperatures of 32F, 50F, and 68F. A plot of the fatigue data for a Shell[2] sulfur modified asphalt mixture (Thermopave) is presented in Figure 5.5. The stiffness modulus for the Thermopave (6×10^5 psi) mixture was approximately two-thirds of the stiffness modulus of an asphaltic concrete (9×10^5 psi). This reduction in stiffness could cause an increase in tensile strain; therefore, facilitating a thicker pavement design.[2]

Fatigue testing of sulfur asphalt mixtures by other researchers[4] indicates that the sulfur asphalt mixtures may have a longer fatigue life than a conventional asphalt concrete. The fatigue testing conducted by Kennepohl, Logan, and Bean[4] was on cylindrical test specimens that were prepared by replacing a portion of the asphalt binder with an equal measure of sulfur (the method of mixing used by Gulf Oil Canada Limited). These cylindrical specimens were tested at 70F with a constant stress of 20 psi and a load rate of 0.04 second load and 0.08 second rest. The data, as presented, are inconclusive as to the superiority of sulfur mixtures over conventional asphalt mixtures. Only the mixtures with the same binder content (in this case 6 percent) should be compared because the type and amount of binder will affect the fatigue life by changing the mix stiffness.[5] The fatigue life of the mixtures containing the same binder content are approximately equal; therefore, no conclusion as to the best mixture should be made from the data in Table 5.9.

5.12 PERMEABILITY

Air permeability of some Thermopave mixtures has been found to be in the range of 16×10^{-12} to 16×10^{-13} $in.^2$ This is below 16×10^{-10} $in.^2$ which is considered by Deme[3] to be acceptable as the maximum value in order for a mix to be impermeable. This low value of permeability existed in the sulfur modified asphalt mixtures despite the high air void contents of 11 percent.

Permeability measurements were also made in the field by Hammond, Deme, and McManus.[2] These permeability measurements were made using the greased ring method (Tentative Method for Measuring the Permeability of Bituminous Mixes, E. Zube, California Division of Highways). "An average permeability reading of 7 ml/min was obtained which is considered comparable with some asphaltic concrete pavements tested."[2] The pavement specimens were also found to contain a high air void content. The void content was determined by ASTM test method C 457 according to the linear transverse method. The microscopic examination revealed that the sulfur in the mixture had entrapped the air in the voids, sealing some of the voids and reducing the permeability.[2]

USE OF SULFUR MODIFIED ASPHALT

The newer generation of sulfur modified asphalt pavement has been placed in Canada. In 1970, a sulfur modified paving mixture called Thermopave, which was developed by Shell Canada Limited,[2] was placed in Richmond, B.C. The traffic count was 9000 vehicles per day with 11 percent trucks. The mix composition was 6 percent of an 85/100 penetration grade asphalt, 80.5 percent medium-coarse sand, and 13.5 percent sulfur. The mixture was placed over a weak subgrade with a high water table. After four years, "the pavement has not shown any detrimental performance."[3]

Other field installations of the same type mixture were placed over an old pavement in Oakville, Ontario, in 1971. Also constructed in Ontario in 1972 was a 1-mile test road to evaluate the structural properties of sulfur-asphalt-sand bases of various thickness.[3] Performance data on these test installations have not been obtained.

Sulfur paving mixtures, prepared by the Gulf[4] method, were placed in Port Colborne, Ontario; Blue Ridge, Alberta; and Windfall, Alberta.

The Port Colborne test section was placed at the entrance to a quarry. Its purpose was to test the feasibility of full-scale production; therefore, no test results were given

The Blue Ridge installation was a full-depth pavement. It was constructed in the fall of 1974 and no performance data are available. A layout of the construction and binder composition is given in Figure 5.6.

In the fall of 1974, a sulfur modified asphaltic overlay was placed on a 1-mile section of Alberta Highway 43 near Windfall, Alberta. The overlay thickness design consisted of 0.5 mile of 2 in. and 0.5 mile of 4 in. of material. A conventional asphalt concrete was placed in the southbound lane and a sulfur modified asphalt mixture was placed in the northbound lane. The sulfur mixture used a 7 percent SA binder, with 40 percent sulfur and 60 percent asphalt. Additional data, such as traffic count, temperatures, and performance were not available.

5.13 EASE OF CONSTRUCTION

The sulfur modified mixtures prepared by the Gulf[4] process are placed using the same construction equipment and methods that are used for a conventional asphalt concrete. Special equipment for storing, heating, and mixing sulfur will be required at the mixing plant, but no special transportation equipment will be required.

Sulfur mixtures prepared by the Shell[1,2,3] process can be placed either of two ways. The mixture can be placed with a conventional asphalt paver, or the mixture can be poured berween forms. In either case, the sulfur modified asphalt mixture is not compacted with a roller. Compaction is accomplished with vibrating screeds while the mixture is still hot (260F). Full-scale production of sulfur modified asphalt mixtures can be adapted to batch hot-mix plants by adding a sulfur handling system that is similar to asphalt cement systems.

5.14 EASE OF MAINTENANCE

Repair of sulfur modified mixtures is similar to that of a conventional asphalt concrete. The Thermopave[2] type mixtures will not require compaction after a repair is made and the sulfur modified mixtures can be opened to traffic as soon as the mixture has cooled.

5.15 ENVIRONMENTAL COMPATIBILITY

Both Shell Canada Limited[3] and Gulf Oil Canada Limited[4] studied part of the environmental problems of using a sulfur type mixture. Gas emission testing was conducted at constructed sites, production plants, and on transportation vehicles. The conclusion of the emission testing is that fume concentrations increase with temperature and decrease a short distance away from the source. To avoid high concentrations of hydrogen-sulfide, the temperature should be kept below 300F. Temperature in excess of 300F would also cause difficulty in mixing the sulfur and asphalt together. Table 5.10 gives the data on the emission testing for sulfur modified asphalt mixtures.

5.16 AVAILABILITY AND COST

Some sources of information[1,2,3] indicate that there is a large stockpile of sulfur. A telephone survey in July 1975 indicated that sulfur is presently being used in great quantities, but future supplies are expected to icrease.

In a telephone survey conducted in February 1974,[1] the price of liquid sulfur was $20/ton. In July 1975, the cost of one dark liquid sulfur was $57.50/long ton F.O.B. New Orleans. This places the cost of sulfur slightly less than the cost of asphalt cement. The price of sulfur varies with locality and supply and generally ranges from $30 to $60/ton.

REFERENCES

CHAPTER 5

SULFUR MODIFIED ASPHALTS

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2. Hammond, R., Deme, I., and McManus, D., "The Use of Sand-Asphalt-Sulfur Mixes for Road Base and Surface Applications," Nov 1971, Shell Canada, Limited, Presented at the annual meeting of the Canadian Technical Asphalt Association.
3. Deme, I., "Processing of Sand-Asphalt-Sulfur Mixes," Proceedings, Association of Asphalt Paving Technologists, Vol 43, Feb 1974, pp 464-490.
4. Kennepohl, G. J. A., Logan, A., and Bean, D. C., "Conventional Paving Mixes with Sulfur-Asphalt Binders," Feb 1975, Gulf Oil Canada, Limited, Presented at the annual meeting of the Association of Asphalt Paving Technologists.
5. Pell, P. S., "Characterization of Fatigue Behavior," Source unknown, University of Nottingham, England, pp 49-64.

Table 5.1

Marshall Properties of Sand Mixtures (150/200 Pen Asphalt) for Sulfur Modified Asphalts

Percent Sulfur	Percent Asphalt	Bulk Density, g/cc	Marshall* Stability, lb	Marshall Flow, 0.01 in.
8	6	1.985	946	6
10	6	2.026	1212	6
12	6	2.063	1954	6
14	6	2.048	2658	6
16	6	2.058	3105	8

* Specimens prepared with two-hammer-blow compaction.

Table 5.2

Marshall Test Data with a Sulfur Asphalt Binder[4]

Binder Content % wt	Air Voids, %		Stability, lb		Flow, 0.01 in.	
	AC*	SA**	AC*	SA**	AC*	SA**
6.0	4.8	7.3	1811	3934	9.5	10.0
6.5	3.4	5.6	1833	4351	10.0	8.5
7.0	2.1	4.5	1398	4362	11.0	8.5
7.5	1.0	4.0	1224	4224	14.0	8.5
8.0	0.6	1.4	1011	4209	15.0	9.5

* 85-100 penetration grade asphalt cement.

** SA binder containing 50% weight elemental sulfur and 50 percent weight 85-100 penetration grade asphalt.

Table 5.3

Aggregate Gradation for Sulfur Asphalt Concrete Mixtures[3]

ASTM Sieve Size	Cumulative Percent Passing				
	Fine Sand	Medium Coarse Sand	Coarse Sand	Low Stone Gravel	High Stone Gravel
3/4 in.				100	100
1/2 in.				95	90
3/8 in.			100	90	85
No. 4			98	85	70
No. 8		100	89	75	65
No. 16		97	74	59	55
No. 30	100	80	46	36	34
No. 50	75	48	20	16	11
No. 100	20	14	6	9	6
No. 200	5	2.5	2	7	5

Table 5.4

Flexural Test Results of Sulfur Modified Asphalt[3]

Temperature, F	Testing Machine Crosshead Speed, in./min	Bulk Density, g/cc	Flexural Strength, psi	Flexural Strain
50	0.008	2.034	126	0.004
50	0.797	1.972	415	0.003
68	0.008	2.015	46	0.006
68	0.787	1.076	162	0.004

Table 5.5

Direct Tensile Results of Sulfur Modified Asphalt Mixture[3]

Temperature, F	Testing Machine Crosshead Speed, in./min	Bulk Density, g/cc	Flexural Strength, psi	Flexural Strain
50	0.008	1.968	73	0.004
50	0.787	1.961	282	0.002
68	0.008	1.972	31	0.003
68	0.787	1.968	79	0.006

Table 5.6

Results of Tension Testing of Sulfur Modified Asphalt Mixtures[1]

Temperature, °F	Deformation Rate, in./min	Asphalt, %	Sulfur, %	Stiffness, psi	Failure Stress, psi	Failure Strain
(a) Using Sand I						
20	0.02	6	13.5	3,550,000	320	0.025
	0.20	6	13.5	6,560,000	334	0.014
	2.0	6	13.5	5,640,000	397	0.024
73	0.02	6	13.5	190,000	38	0.153
	0.2	6	13.5	206,000	91	0.174
	2.0	6	13.5	460,000	179	0.240
	2.0	4	13.5	520,000	258	0.101
	2.0	2	13.5	790,000	272	0.062
	2.0	4	16	1,025,000	207	0.080
	2.0	2	16	1,480,000	290	0.048
	2.0	4	20	1,455,000	233	0.062
	2.0	2	20	2,005,000	325	0.041
135	0.2	6	13.5	15,000	5.21	0.211
	2.0	6	13.5	43,000	17.50	0.190
(b) Using Sand II						
20	0.02	6	13.5	1,100,000	275	0.026
	0.20	6	13.5	2,730,000	283	0.018
	2.0	6	13.5	3,530,000	432	0.010
73	0.02	2	13.5	493,000	118	0.063
	0.02	4	13.5	125,000	30	0.116
	0.02	6	13.5	107,000	14	0.112
	0.02	6	16	200,000		0.16
	0.02	6	20	240,000	35	0.13
	0.20	4	13.5	260,000	83	0.100
	0.20	6	13.5	260,000	46	0.166
	0.20	6	16	310,000	61	0.117
	0.20	6	20	380,000	68	0.110
	2.00	2	13.5	624,000	258	0.055
	2.00	4	13.5	353,000	176	0.155
	2.00	6	13.5	327,000	123	0.205
	2.00	6	16	867,000	155	0.127
	2.00	6	20	1,080,000	182	0.103
135	0.20	6	13.5	6,000	1.24	0.059
	2.00	6	13.5	36,000	15.2	0.352

Table 5.7

Tension Test Results of Asphaltic Concrete Containing 6.2% Asphalt[1]

Temperature, °F	Deformation Rate, in./min	Stiffness, psi	Failure Stress, psi	Failure Strain
20	0.02	5,200,000	491	0.11
	0.20	5,300,000	412	0.015
	2.00	6,000,000	439	0.016
73	0.02	137,000	81	0.427
	0.20	255,000	148	0.53
	2.00	415,000	386	0.33
135	0.20	4,000	8.4	0.814
	2.00	14,000	14.0	0.958

Table 5.8

Skid Resistance of Sulfur Modified Asphalt Pavement Using the British Road Research Laboratory Pendulum Tester[2]

	Skid Resistance Reading
Sulfur Modified Asphalt Pavement After 9 Months	64.8
Asphaltic Concrete Surface After 1 Year	55.1

Table 5.9

Fatigue Test Results of Cylindrical Specimens of Sulfur Modified Asphalt[4]

Sample Description	Load Applications to Failure × 10^6	
	Test No. 1	Test No. 2
Test specimen with 6.0% wt asphalt	2.3	1.87
Test specimen with 6.7% wt S/A binder 25% wt sulfur	1.0	8.5*
Test specimen with 6.0% wt S/A binder 50% wt sulfur	2.6*	2.7
Test specimen with 7.4% wt S/A binder 50% wt sulfur	3.6*	0.8

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* Test stopped before failure occurred.

Table 5.10

Comparison of Emissions of Asphalt and Sulfur Modified Asphalt Binders[4]

Pollutant	Asphalt	SA Binder
H_2S; ppm	0.2	1.8
SO_2; ppm	0.1	0.2
RSH, COS, CS_2; ppm	N.D.* < 0.1	N.D.* < 0.1
Total hydrocarbon	6	6
H_2S (with Kitagawa tube) ppm	N.D.* < 0.1	2-3
SO_2 (with Kitagawa tube) ppm	N.D.* < 0.1	0.5

* N.D. = Not detected.

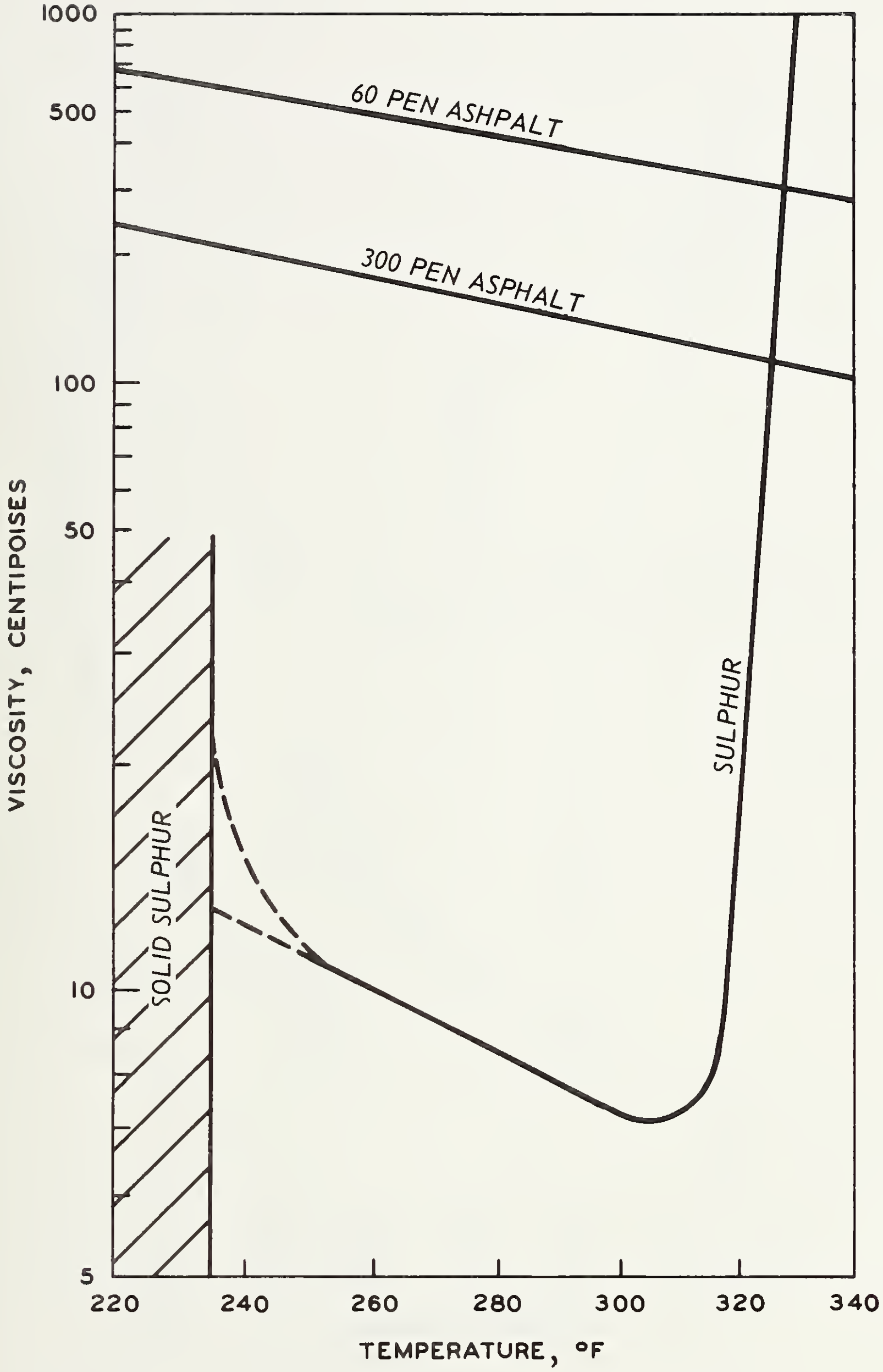

Figure 5.1. Viscosity-temperature relationships for asphalts and sulphur[3]

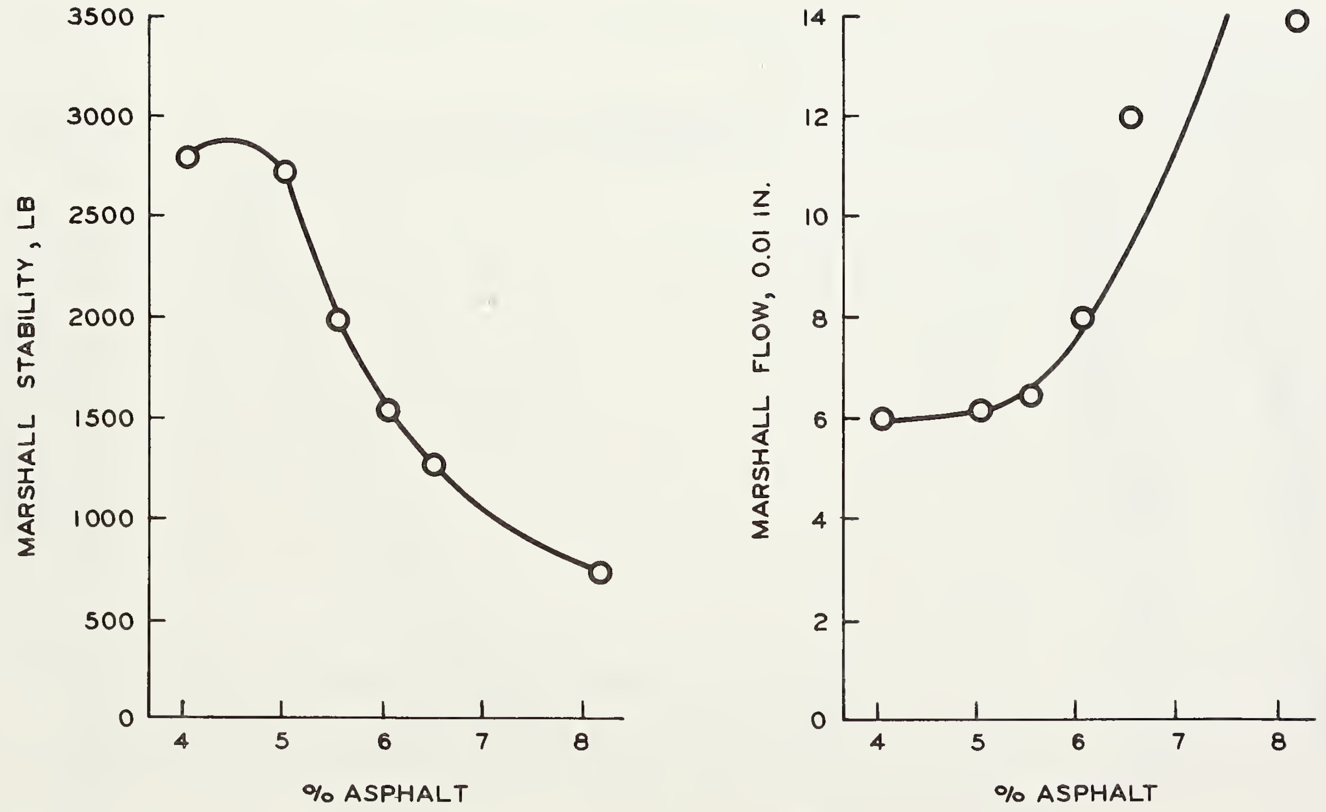

Figure 5.2. Marshall properties for sulfur modified asphalt mixtures with 13-1/2 percent sulfur[2]

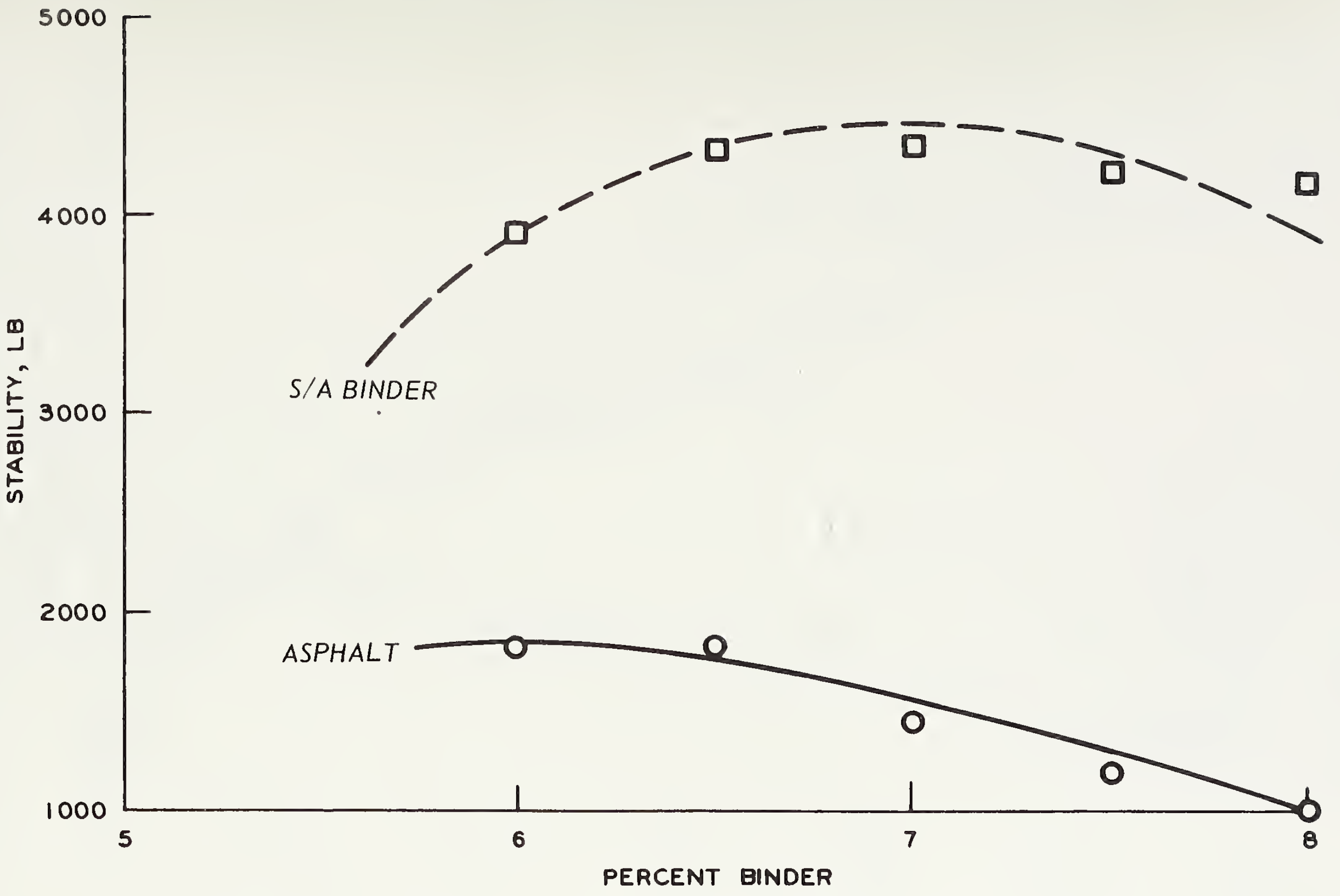

Figure 5.3. Effect of binder content on Marshall stability of sulphur modified asphalt[4]

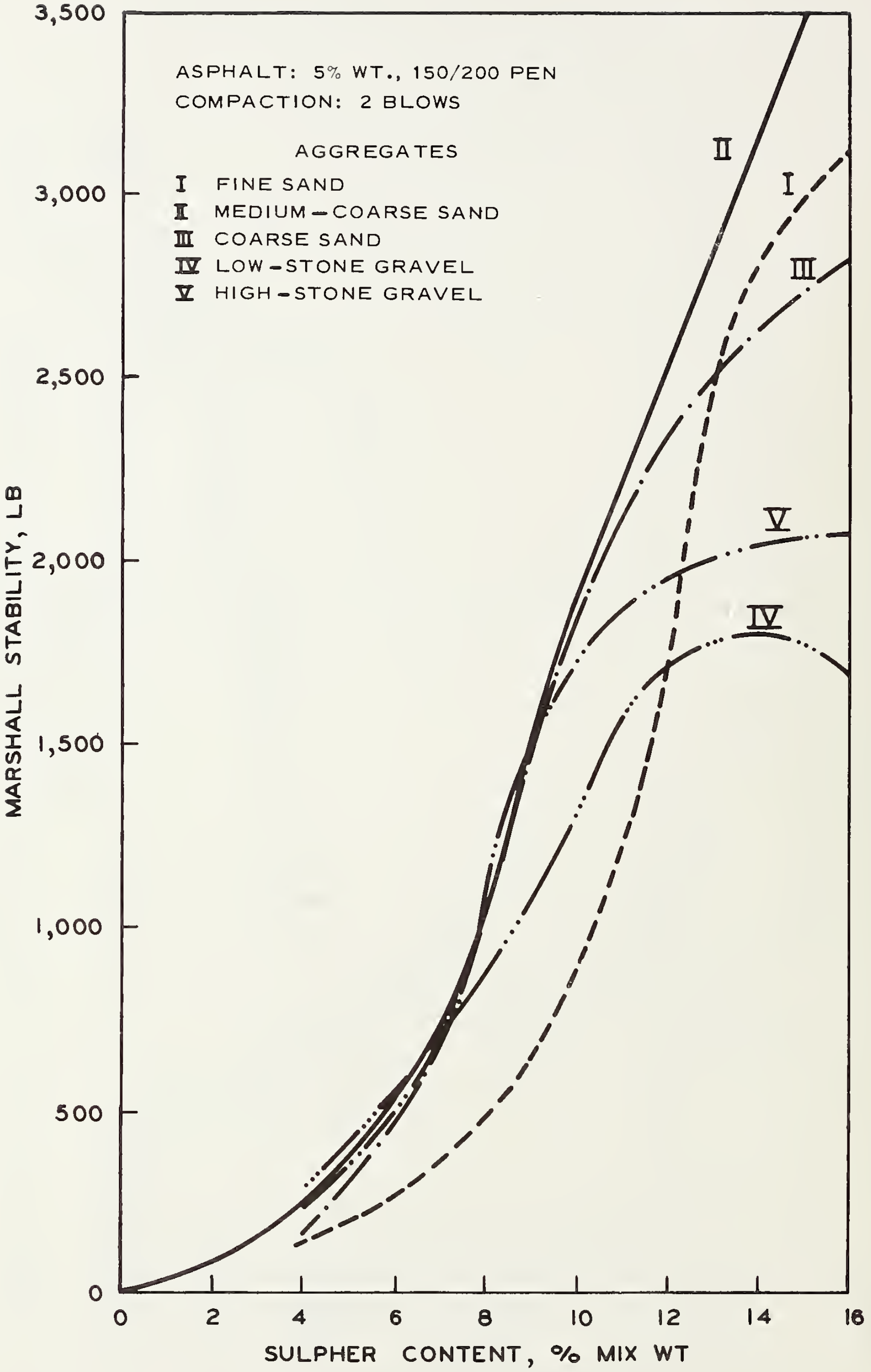

Figure 5.4. Effect of sulfur content on Marshall stability for mixtures prepared using various aggregate gradations (shell method of mixing)[3]

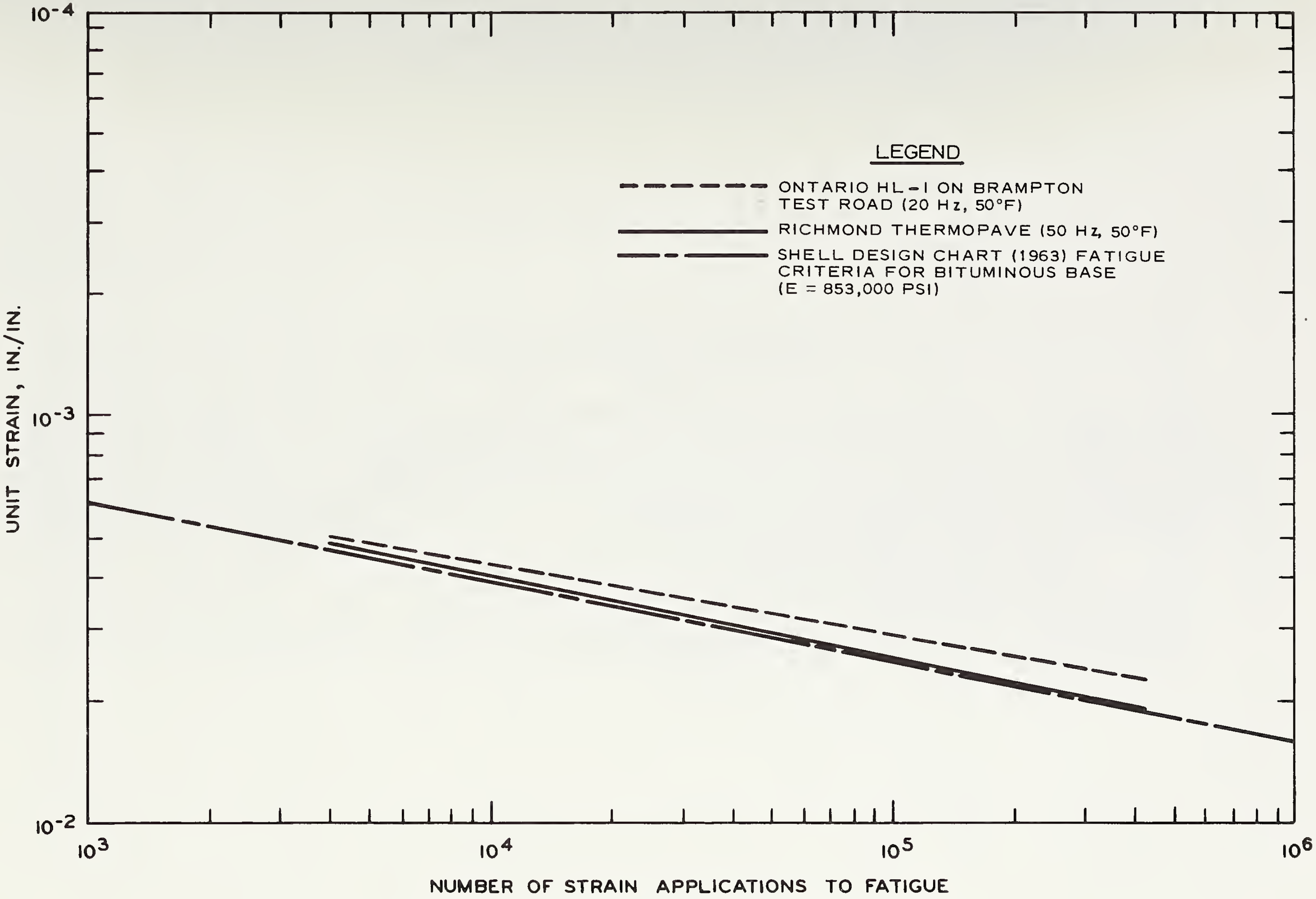

Figure 5.5. Fatigue life data for a sulfur modified asphalt (three-point bending at constant deflection)[2]

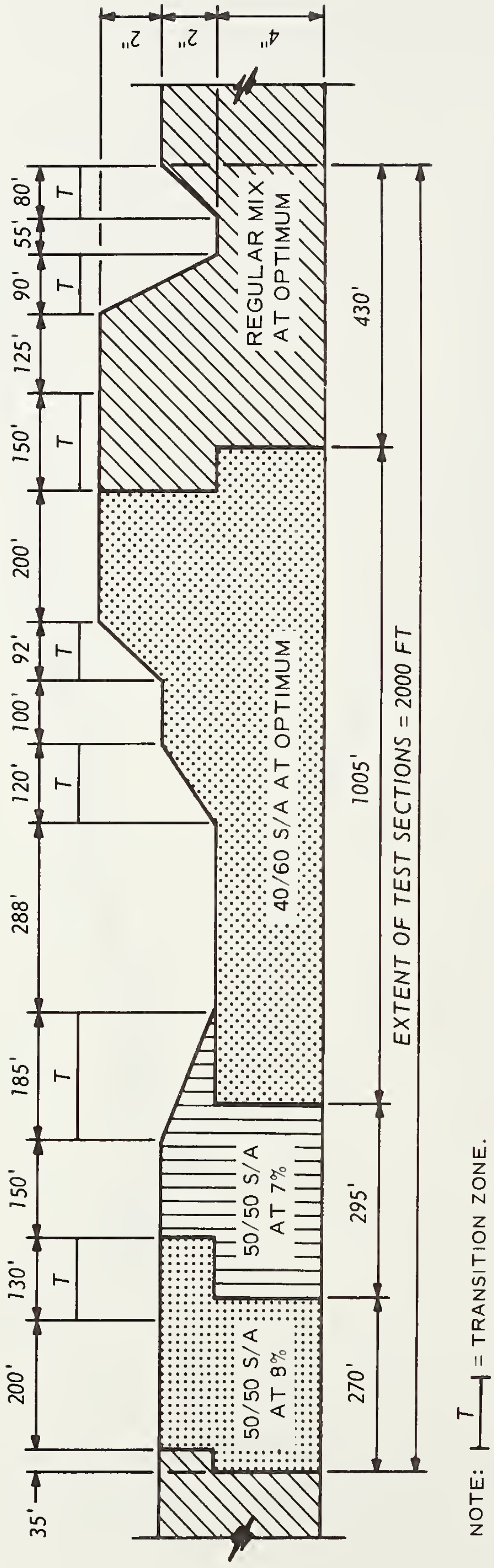

Figure 5.6. Layout of Blue Ridge sulfur modified asphalt construction[4]

CHAPTER 6

NONCALCAREOUS INORGANIC CEMENTS

INTRODUCTION

6.1 BACKGROUND

Numerous materials can be classified as noncalcareous inorganic cements. These include sulfur, phosphate cements, sorel cement, (i.e., magnesium oxychloride cement), sodium silicate, the chlorides $NH_4\ Cd\ Cl_3$ and $K_2\ SnCl_6 \cdot 2\ H_2O$, and others. A literature review was made to see which of the materials fitting this classification could be considered as having potential to replace conventional portland cement concrete or to improve one or more of its properties by their use with it. Sorel cement was rejected from serious consideration since it is damaged by water.[1,2] No promising information with regard to the desired use of sodium silicate or chloride cements was found. They are generally considered too specialized to be useful for pavement work and were not investigated further. The bulk of the review was therefore confined to sulfur and to phosphate cements. Several of the phosphate references that were examined mentioned that sodium silicate is a common refractory cement whose bonds are not as strong as the phosphate bond. This was taken as added evidence that a specific search for references pertaining to the use of sodium silicate as cement was not necessary.

While there has been some work with sulfur in the present context for about 50 years, the major emphasis came about in the last 10 years because sulfur was becoming cheaper as stock piles of it accumulated, especially in Canada, where it was a by-product of cleaning natural gas. The price advantage that sulfur enjoyed largely disappeared by 1975 however, so any use will need to be based on a quality rather than a price basis. The data on phosphate cements is the result of continuing efforts by industry to improve their use as refractory cements or in patching formulations for refractory linings.

6.2 DESCRIPTION OF MATERIALS OR SYSTEMS

6.2.1 Sulfur. Sulfur is used in many different ways and many references on these uses are available. For simplicity, the various uses have been categorized as follows with the references for each use being appropriately grouped in the reference listing for this chapter:

a. Sulfur (elemental) - General (SG).[3-10]

b. Sulfur - impregnated or infiltrated concretes (SIC).[11-17] These materials are formed by the treatment of hardened portland cement mortar or concrete specimens by vacuum, followed by immersion in molten sulfur which may or may not be under pressure to help drive the sulfur into the specimens and then subsequent exposure to atmospheric pressure and normal temperature ranges to allow the sulfur to harden.

c. Sulfur - surface applications (SSA).[18-26] This category differs from the above in that no vacuum is used; concrete specimens are dipped into liquid sulfur, or it is applied to surfaces as a spray or with a brush with the intent that the liquid sulfur penetrates as far as possible.

d. Sulfur - Admixture (SAD).[27] This unusual category deals with the addition of powdered sulfur to portland cement pastes or moratrs before they are mixed with water. Specimens are then autoclaved or otherwise heated, to liquefy the sulfur. The intent is that it then forms a continuous phase with subsequent cooling.

e. Sulfur concrete (SC).[28-46] This material is a combination of aggregates and molten sulfur without any portland or other type of cement; the sulfur serves as the binder when the mixture cools and the sulfur solidifies. This material is not intended to be a substitute for portland cement concrete but may possibly be preferable in some specific applications, probably involving limited volumes.

f. Sulfur Foam (SF).[47-51] This category covers the use of foamed sulfur or sulfur formulations for insulation in construction activities. There have been indications from other work* that coating porous aggregates

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*Buck, A. D., "Recycled Concrete," US Army Engineer Waterways Experiment Station, Miscellaneous Paper C-72-14, May 1972, Vicksburg, MS.

before use in concrete may improve their frost resistance. Doing this by dipping porous aggregates in molten sulfur is an attractive concept that does not seem to have been tried.

6.2.2 Phosphate Cements. The information gathered on these materials has been categorized as follows with the references for each category being appropriately grouped in the reference listing for this chapter:

a. Phosphorous - General (PG).[52-54] Information on the chemistry of phosphorous compounds is readily available, but it generally does not contain specific information as to the actual mechanism of the setting of phosphate cements or their behavior.

b. Phosphate cements (PC).[55-76] These materials combined with suitable aggregates were originally used as refractories or as patching material for refractory linings. These same materials are now finding a market for making quick-setting, high-strength patches for concrete but do have a number of limitations.

MECHANICAL PROPERTIES

6.3 SULFUR

6.3.1 Sulfur (Elemental). Dale,[8,9,10] Ludwig,[9,10] and Thaulow[11] have discussed the properties of sulfur and the alterations to these properties that can be made by the use of additives. Table 6.1 summarizes the methods of altering the properties of sulfur. The ability to change sulfur from a material that supports combustion to one that will burn but will not support combustion is perhaps the more important of these possible changes. Since interest was in materials that might be used in addition to or instead of portland cement concrete, emphasis was on obtaining data that were comparative with conventional concrete.

6.3.2 Sulfur-Impregnated or Infiltrated Concrete. Thaulow[11] and Malhotra, et al.,[12,13,16] have dealt with this process in detail. A tabulation of their results compared to control concretes is given in Table 6.2. It can be seen that strength, modulus of elasticity, and resistance to freezing and thawing all increase with sulfur impregnation while porosity decreases. Thaulow[11] got an increase by a factor of nearly 3 to 20,000 psi in his work while Malhotra's best factor was 10 to 10,000 psi. Thaulow[11] used a mature, stronger concrete than Malhotra, thus Thaulow's concrete contained less void space that could be filled with sulfur, and it is the filling of this void space that is believed to be responsible for the improvements that have been found. Unpublished data from the Waterways Experiment Station (WES) on very poorly consolidated concrete, dried and immersed at 48 hr age in a molten cylinder capping compound containing approximately 25 percent sulfur, upon hardening also experienced a strength increase by a factor of 10. The amount of sulfur that was taken into the specimens of Thaulow and Malhotra ranged from about 8 to 13 percent of the dry weight of the concrete. Other factors were not evaluated in these references. There is need for additional data, especially data covering service records or similated service records to verify that the improvements are long lasting. It is interesting to note that in 1940 Kobbe[25] had already done some vacuum and pressure treatments of concrete with sulfur, but he did not provide any data in this reference.

6.3.3 Sulfur-Surface Applications. This process is just a simpler version of the SIC process, but without use of vacuum. Therefore, as expected the properties of the treated concrete or mortar are affected like those in Table 6.2. Comparative data are shown in Table 6.3. Kobbe[25] states that concrete absorbs about 15 percent sulfur by weight during immersion. This value seems a little high, but similar, to those quoted by Thaulow[11] and by Malhotra, Painter, and Soles.[12] Other kinds of data were not evaluated against conventional concrete or mortar. Again, long-term data are needed to verify that improvements are permanent. Kobbe[25] notes that fire hazard with sulfur-impregnated concrete is very slight and states that the treated concrete will not support combustion. Enough heat will, however, cause the sulfur vapors to be driven off. The sulfur referred to by Kobbe[25] is sulfur before fire-retarding additives became known.[9] His remarks should apply equally well to the SIC category.

6.3.4 Sulfur Admixture. Beaudoin and Sereda[27] added different amounts of powdered sulfur to portland cement and to mortars of portland cement and sand before they were mixed with water. The resulting pastes and mortars were then autoclaved and tested. They found an increase in compressive strength for the paste with 2 percent sulfur and a decrease for the mortar with siliceous sand (Table 6.4). This effect needs additional investigation. There was also an increase in strength when the sand was limestone (Table 6.4). The addition of sulfur increased the surface area of the paste, caused a change from a coarse to a fine pore size distribution, increased drying shrinkage, decreased density and porosity, and increased microhardness. However, these properties were still similar to those for normal hydrated pastes when an optimum amount of sulfur (2 percent) was used. Kobbe[25] found that the addition of sulfur powder to a mixture had little or no effect on strength even if the sulfur is caused to melt. As he neither mentions the amount of sulfur or method of heating, his staement remains enigmatic when compared to Beaudoin and Sereda's data.[27]

6.3.5 Sulfur Concrete. Figure 6.1 shows a comparison of mixture proportions used in sulfur concrete and in portland cement concrete. Figure 6.2 shows strength gain of the two kinds of concrete with time. Their creep behavior is compared in Figure 6.3 and stress-strain curves are shown in Figure 6.4. Figure 6.5 shows the effect on the durability of sulfur concrete of different amounts of pyrite and conventional sand. All of these figures were taken from Loov[45,46] or Loov, et al.[29] Sulfur concrete gains most of its compressive strength (85 percent) during the first few hours. This gain is much more rapid that conventional concrete, but the final result is similar (Figure 6.2). Creep of sulfur concrete is several times greater than conventional concrete (Figure 6.3) which may or may not be an advantage. The stress strain curves in Figure 6.4 indicate that the sulfur concrete is brittle compared to conventional concrete especially above 4350 psi. The modulii of rupture and of elasticity of sulfur and conventional concrete are similar.[45,46] Freeze-thaw resistance of sulfur concrete is poor unless additives are used (Figure 6.5).

Overall, sulfur concrete and portland cement concrete are similar in properties and cost. Each material has its own advantages and disadvantages. Loov[45,46] concluded that sulfur concrete is a potentially useful material for specialized applications, but it will not replace conventional concrete nor should it be expected to do this.

6.3.6 Sulfur-Foam. Rennie[48] provided a general review of rigid sulfur foams and concluded the following about them:

Property	Data
Compressive Strength	54-400 psi
Flexural Strength	Increases with density
Freeze-thaw Resistance	Satisfactory
Thermal Conductivity	0.25-0.45 B.T.U. in./hr ft^2 F. at 86F for 7 to 32 pcf foams. Somewhat higher with higher density foams.

These and other data are shown in Table 6.5. Sulfur foam properties are compared to styrofoam and polyurethane foam in Table 6.6. Dale, et al.,[51] found a curvilinear relationship between compressive strength and foam density with strengths as high as 600 psi occurring at foam densities from 40 to 60 pcf. They also found that shrinkage of the foams upon solidification was negligible. Direct application of sulfur foams in a zero maintenance highway concept seems unlikely at this time. If any application evolves, it will probably be in Canada and farther north as some form of insulation against cold placed under the pavement.

6.4 PHOSPHATE CEMENTS

References 55 through 76 deal with phosphate cements. Only references 56 and 62 deal with the application of phosphate cement formulations for use as concrete patches. The others generally have to do with phosphate cements in connection with refractories. Janowiecki and Semler[56] evaluated a silico-phosphate cement combination as rapid patching material for repair of concrete pavements and bridge decks. Their silicate aggregate was the mineral wollastonite ($CaSiO_3$) and their cement was buffered phosphoric acid. They found that compressive strengths of 5000 psi or more could be obtained in one hour and that other properties were equal to or better than those of portland cement concrete. Therefore, they suggested that this material should be field tested for use as a concrete patching compound. Unpublished WES data for Republic Steel's quick setting concrete[62] verified the claim of quick setting and high early strength for that product. This material took between 8 to 24 hours to reach a compressive strength of 5000 psi. It is an acidic ammonium phosphate solution with an aggregate that is periclase (MgO) plus dolomite ($CaCO_3 \cdot MgCO_3$). The properties of the material tested by Janowiecki and Semler[56] and by WES seem generally similar. Republic Steel licensed their product to the following four producers:

Company	Trade Name
USM Corp., Upco Chemical Division	Bostik 275
W. R. Grace and Co., Construction Products Division	Derex 240 Concrete
Steelcote Manufacturing Co.	FC 100 (Hi-Strength Concrete)
Acme Highway Products Corp.	(Acmaset)

USE OF SULFUR

6.5 EASE OF CONSTRUCTION

6.5.1 Sulfur-Impregnated and Infiltrated Concrete. There should be no difficulty in the use of this process for precast concrete elements. Some problems are foreseen for its application to cast-in-place pavements, but these problems are similar to those associated with the polymer impregnation of pavements (Chapter 10). As some pavements and bridge decks have been successfully impregnated with polymers, it is believed that a successful system for sulfur impregnation or infiltration could be devised.

6.5.2 Sulfur-Surface Application. There is no special difficulty with the use of this process; it can be used anywhere by simply brushing or spraying molten sulfur on any desired surface.

6.5.3 Sulfur-Admixture. Difficulty will probably be experienced in providing enough heat to melt the admixed sulfur in a field installation. It would have to be done at such a time so that the concrete could withstand the thermal conditions.

6.5.4 Sulfur Concrete. There are no serious difficulties involved here. Provision must be made to liquefy the sulfur. There would be the usual small problems associated with the use of any new process.

6.5.5 Sulfur-Foam. This should not pose any special problems as the equipment necessary for foaming and applying the water has been developed and a number of successful jobs completed.

6.6 EASE OF MAINTENANCE

There should be no special maintenance problems for any of the five sulfur categories.

6.7 ENVIRONMENTAL COMPATIBILITY

There is possibility of air pollution and personal harm or discomfort to humans with any sulfur usage since fire could create noxious fumes.

6.8 AVAILABILITY AND COST

As mentioned earlier, there was a time recently when sulfur was somewhat of a low-cost waste by-product. The present status is that there are and will be adequate supplies of sulfur, but it will no longer be a cheap item.[7] This latter point is illustrated by a change in price in Calgary, Alberta, Canada, from $6.40/ton in July 1971 to about $25/ton in December 1974.[45,46]

USE OF PHOSPHATE CEMENTS

6.9 EASE OF CONSTRUCTION

These cements can be used as either cold setting or hot setting materials. The two systems described in this report (phosphoric acid with silicates or oxides) are both cold setting processes that do not use water. With either system the extremely rapid set (5 to 10 min) can cause handling problems. The ammonia fumes released when the Republic Steel product is mixed are objectionable to working personnel. Phosphate cement systems can be formulated with water as a lubricant; however, it is then necessary to fire the mixture to remove the added water. This would be difficult in the field. The need for heating if water is added and the need to avoid water with the cold setting mixtures makes it clear that phosphate cement formulations are not compatible with fresh portland cement concrete. They are compatible with hardened portland cement concrete.

6.10 EASE OF MAINTENANCE

Phosphate cement systems are not expected to have any unusual maintenance problems.

6.11 ENVIRONMENTAL COMPATIBILITY

No environmental problems due to the use of phosphate cements are anticipated other than the small amount of ammonia gas already mentioned. It is not considered serious for the small amounts of phosphate cements or for work out of doors where it can escape readily.

6.12 AVAILABILITY AND COST

Information on both recent availability and cost was not readily available, although indications are that it is quite expensive. Availability should not be a problem as long as usage is restricted to patching applications. Small quantity usage may also be dictated by cost.

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Table 6.1[18]

Methods of Altering the Properties of Sulfur

Characteristics to be Altered	Class of Additives	Example Materials	Amount Used (wt %)
Mechanical properties	Plasticizers	Polyethylenetetrasulfide	1-10
		Liquid polysulfides (Thiokol-LP3)	1-10
		Styrene monomer	1-25
		Phosphorus pentasulfide	1-10
	Fillers	Limestone	5-70
		Clay	1-15
		Talc	1-15
	Fibers	Glass	1-5
		Asbestos	1-5
Fire resistance	Fire retardants	Chlorowax-70S-dipentene dimercaptan	1-10
	Crosslinking agents	Dicyclopentadiene	0.5-3
Viscosity	Chain terminators	2-Mercaptobenzothiazole	0.5-1
Corrosion inhibitor	Rust inhibitors	Lead chromate	0.5-2
Bacterial resistance	Bactericides	Sodiumpentachlorophenate	0.5-1
Color	Dyes	Oil orange	0.5-1
	Pigments	Carbon black	0.5-5

Table 6.2

Sulfur Impregnated Concrete Properties

References	Properties--Compared to Conventional Concrete*							
	Compressive Strength	Splitting Tensile Strength	Modulus of Elasticity, E	Strain at Rupture	Density	Freeze-Thaw Durability**	Poisson's Ratio	Porosity
Thaulow[11]	2.7 (2-1/2 yr)	--	2+	-1	-1	--	--	--
Malhotra, Painter, and Soles[12]	7.5 (48 hr)	-4-1/2 (48 hr)	2+	--	--	-13 (by cycles)	-1-2	0.6-0.8
Malhotra[13,14]	10	5	2+	--	--	20 (by cycles)	1+	no dat

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* SIC/Control.
** According to ASTM C 666, freeze in air and thaw in water.
\+ Used high quality concrete.
\# Used low quality concrete.

Table 6.3

Sulfur - Surface Applications

References	Properties--Compared to Conventional Concrete or Mortar*			
	Compressive Strength	Tensile Strength	Porosity	Shear Strength
Kobbe[24]	5 (concrete) 2-4 (mortar)	-- 5-10 (mortar)	-0 (concrete) 0.3-0.5 (mortar)	no data
Kobbe[25]	6 (concrete)	5 (concrete)	no data	no data
Dale and Ludwig[26]	no data	no data	no data	2+ (concrete)
Malhotra[13]	5 (concrete)	no data	no data	no data

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* SSA/Control.

Table 6.4

Properties of Sulfur as an Admixture*

	Properties--Compared to Autoclaved Cement Paste or Mortar	
References		Compressive Strength
Beaudoin and Sereda[27]	Autoclaved cement paste	1.75 (2 percent S)
	Autoclaved mortar	0.5 (Siliceous aggr) 1.5 (Limestone aggr)

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* SAD/Control.

Table 6.5

Typical Properties of

Rigid Sulfur Foam[48]

	Method	Density, pcf				
		10		20	30	35[2]
K-factor, Btu In./Hr Ft^2 ^{o}F at 86F	ASTM D 2326	0.28	0.28	0.34	0.44	ND
Coefficient of linear thermal expansion,[3] in./in. ^{o}C (perpendicular to rise) 20 → - 160C	ASTM D 696	3×10^{-5}	4×10^{-5}	2×10^{-5}	2×10^{-5}	ND
Compressive strength, psi at maximum to 10% deformation (parallel to rise)	ASTM D 1621	50-90	40-60	150-200	250-320	350-40
Compressive modulus of elasticity, psi		2000-3000			9,000-12,000	
Flexural strength, psi	ASTM D 790	20-30	50-70[1]	50-60	90-120	ND
Flexural modulus of elasticity, psi		3000-7000	1000-3000			
Water-vapor transmission	ASTM C 355					
Grain/hr ft^2		0.7-1.8	ND	ND	ND	ND
Perm-in.		1.0-2.7	ND	ND	ND	ND
Water absorption, Vol % in 24 hr	ASTM D 2127	1-2	2-3	ND	ND	ND
Vol % in 84 hr	Special, under 20-In. head of water	3-4	ND	1-4	1-2[4]	7-8

(Continued)

Table 6.5 (Concluded)

		Density, pcf				
	Method	10		20	30	35[2]
Freeze-thaw resistance for	ASTM C 290-67	No crack	No crack	No crack	No crack	ND
Closed-cell content, %	ASTM D 1940-62T	10-80	10-80	15-70	20-65	8
Open-cell content, %		15-90	15-90	15-70	10-60	65
Cell wall content, %		5-11	5-11	15-18	23-27	27

ND = Not determined.

[1] Higher flexural strength is obtained at increased cost.

[2] A very low cost high density foam.

[3] Data Limited.

[4] A high closed-cell content foam (88% closed cells adn 12% open cells) absorbed only 2 vol % water after 27 days' immersion under 20-in. head of water. However, water absorption generally increases with time for foams with high open-cell contents.

Table 6.6

Comparison of Foam Properties[48]

Property	Chevron Sulphur Foams				Styrofoam H1-35	Polyurethane Foam
	H_2S Blown	CO_2 Blown				
Density, pcf	15	10	20	30	2.2-3.3	2.2-2.4
Compressive strength, psi at Max. to 10% deformation (parallel to rise)	35	60	150	300	35[2]	40
K-factor, Btu in./hr ft^2 °F at 86F	0.35	0.28	0.34	0.44	0.23[1]	0.16
Flexural strength, psi		20	50	100	85	40
Water vapor transmission						
Grain/hr ft^2	1.3-3.0	0.7-1.8				1.5
Perm-in.	5.7-9.8	1.0-2.7			0.3-1.5[3]	2.1
Water absorption, Vol %	19	1-2	1-2	1	0.25	1-2
Closed cells	5	10	25	40	90	90
Freeze-thaw for 100 cycles	Cracks	No crack	No crack	No crack	No crack	No crack

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[1] At 75F.
[2] At 5% deflection.
[3] Data for Extruded Polystyrene Foam for Modern Plastics Encyclopedia.

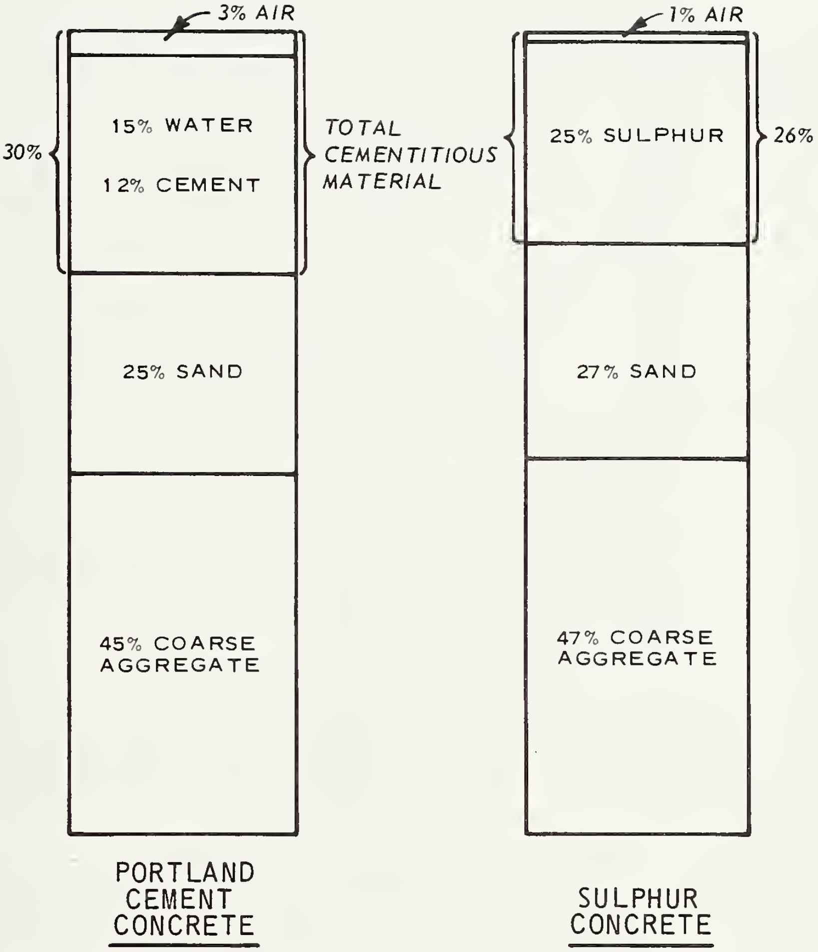

Figure 6.1. Comparison of mixture proportions for sulfur concrete and Portland cement concrete (5000-psi at 24 hours)[12]

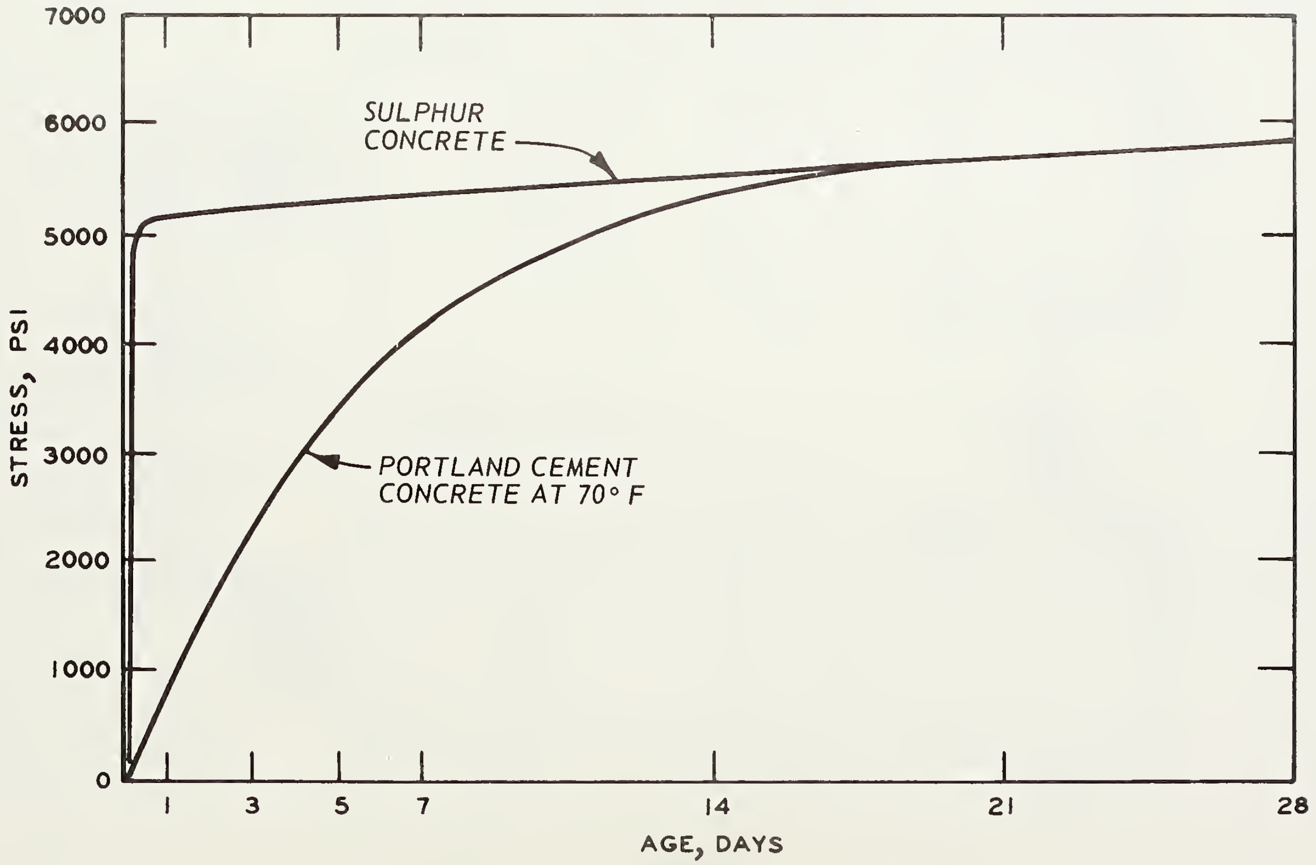

Figure 6.2. Strength gain of sulfur and Portland cement concrete with time[45]

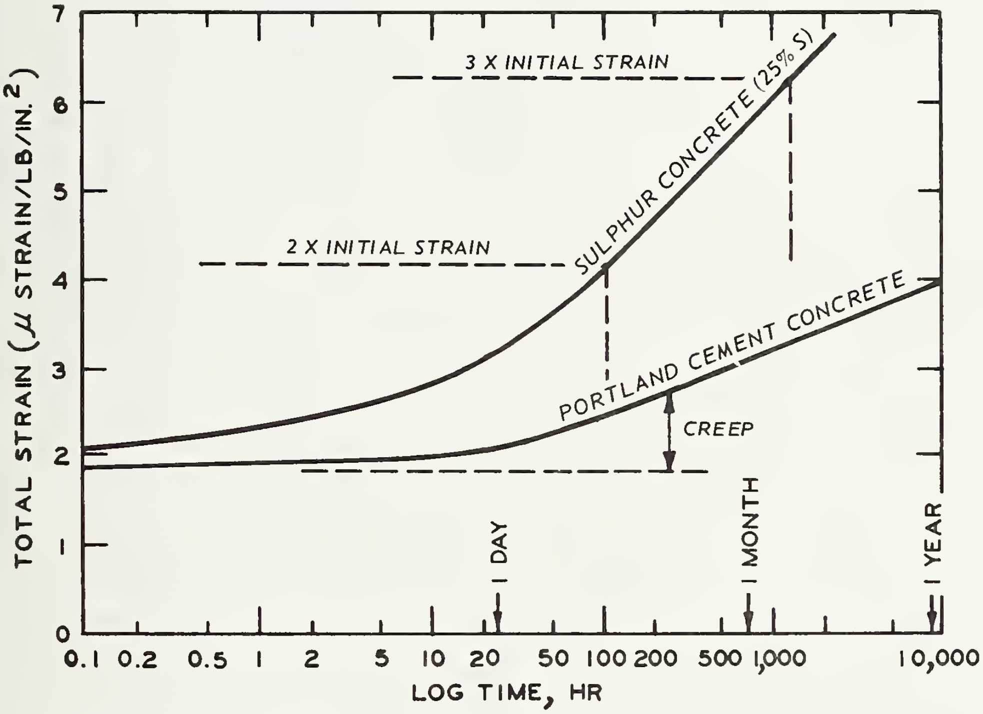

Figure 6.3. Creep behavior of sulfur concrete and a comparable Portland cement concrete at 70F[29]

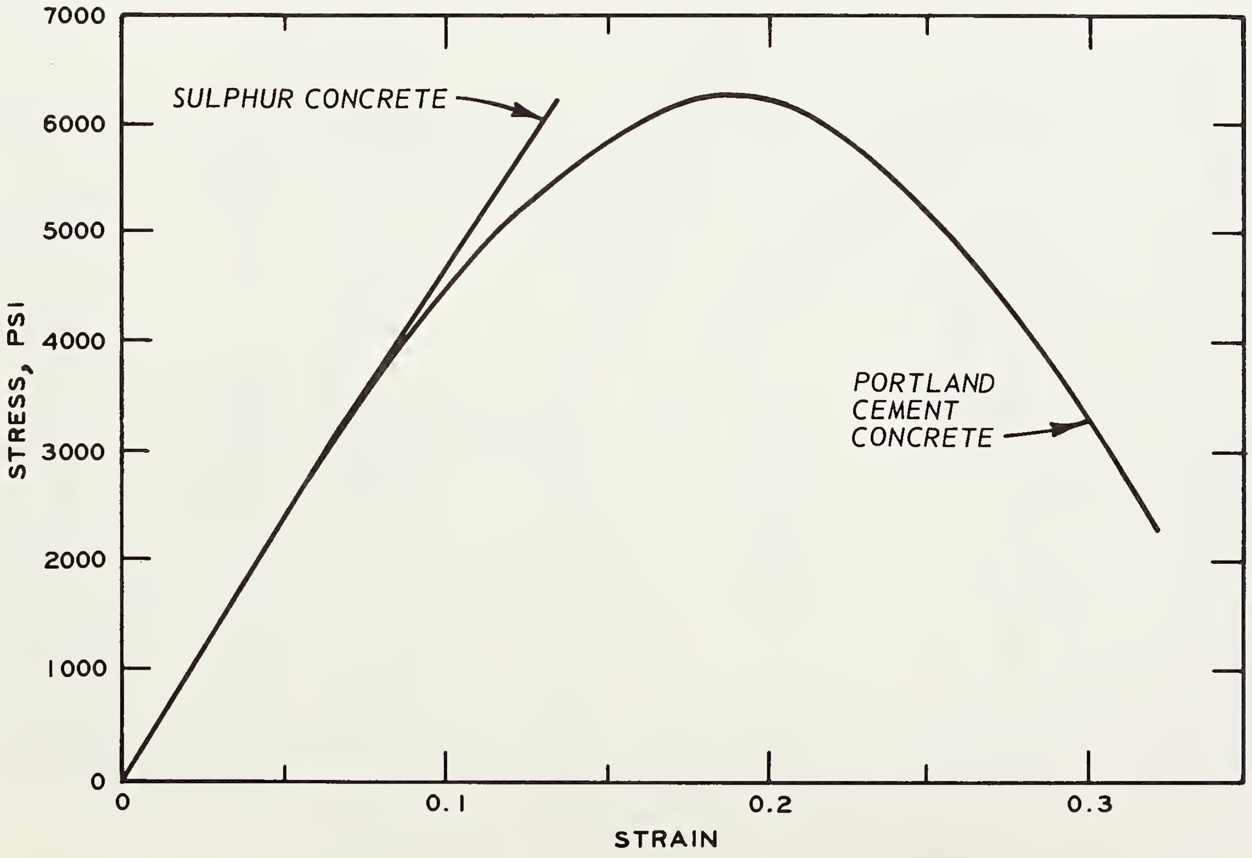

Figure 6.4. Stress-strain curves for sulfur and Portland cement concrete[45]

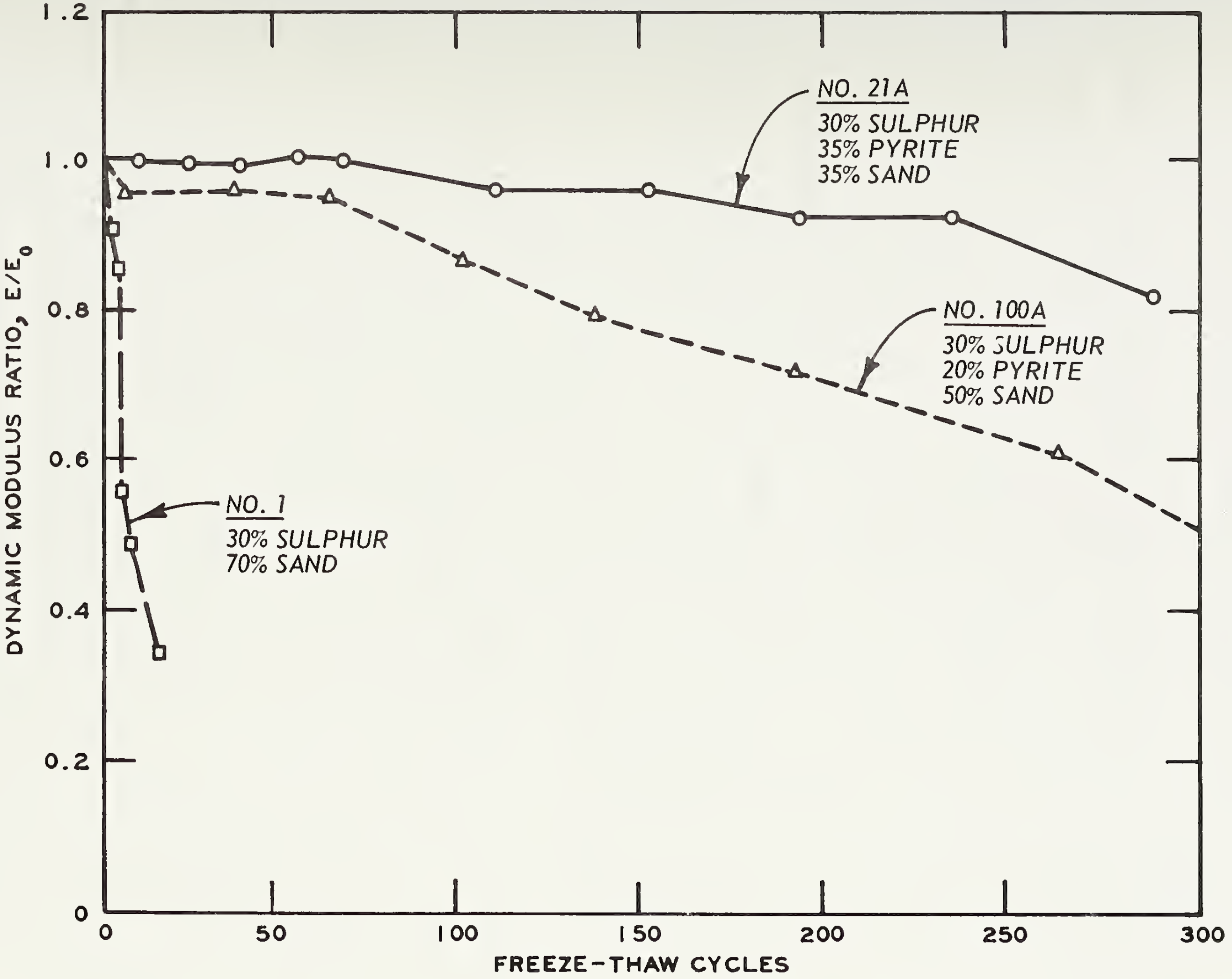

Figure 6.5. Effect of freeze-thaw cycling on dynamic modulus of elasticity of sulfur concrete[41]

CHAPTER 7

PORTLAND CEMENT CONCRETE

INTRODUCTION

7.1 BACKGROUND

The use of cementing materials to bind stone together to form a structural material dates back to the ancient Egyptians and perhaps even before recorded history. Some of this material still survives today. The type of cementing material has varied over the ages and in different locations and societies. In 1824, Joseph Aspdin, a Leeds, England builder took out a patent on what he called "portland cement." This cement was prepared by heating a mixture of finely divided clay and hard limestone in a furnace until carbon dioxide (CO_2) had been driven off. The resulting clinker was then pulverized and used. The prototype of modern cement was made in 1845 by Isaac Johnson who burned a mixture of clay and chalk until clinkering occurred. The name Portland Cement was given to the original set material due to its resemblance in color and quality to portland stone which was a limestone quarried in Dorset, England.

Since these early developments, the use of portland cement has developed rapidly and widely, with portland cement concrete becoming one of the most versatile construction materials available in the world. In the United States, concrete has been and is presently used extensively in pavement construction. Concrete has been repeatedly studied, researched, and tested with innumerable reports, papers, and books being written on all aspects of the materials, concrete properties, performance, and construction techniques. The following sections of this chapter are just a brief glimpse at the information that is available on the various factors or properties being considered for candidate zero maintenance materials. Although portland cement concrete is generally not thought of as a zero maintenance material, much is known about its properties and behavior such that by combining this knowledge with the proper construction practices, a portland cement concrete can be produced that will have greatly improved long-term performance in terms of reduced maintenance over the quality of concrete generally occurring in pavements today.

7.2 DESCRIPTION OF MATERIALS

7.2.1 Portland Cement Concrete. Concrete is a composite material which consists essentially of a binding medium within which are embedded particles or fragments of aggregate.[1] In portland cement concrete, the binder is a mixture of portland cement and water. The aggregate is generally graded in size from a fine sand to pebbles or fragments of stone which may range up to several inches in diameter. The aggregate occupies roughly three-quarters of the volume of typical concrete with the remaining volume being filled with cement, water, and air voids. The air voids may be either entrapped or deliberately entrained or both with the amount varying from 1 to 2 percent for entrapped and up to 8 or 9 percent for both entrapped and entrained. Four to 6 percent total air is common. The overall proportions of the ingredients can be greatly varied to satisfy particular job requirements.

7.2.2 Portland Cement. Portland cement is the product obtained by pulverizing clinker consisting essentially of hydraulic calcium silicates. It usually contains calcium sulfates as an interground addition. The resulting finely powdered substance, usually gray or brownish in color, will react with water to produce new compounds capable of imparting a stonelike quality to the hardened mixture.

Portland cement comes in many varieties and types. These cements include[2] rapid-hardening, quick-setting, white, water-proofed, hydrophobic, low-heat, sulphate-resisting, Kuhl, Iron-ore (Erz), Ferrari, expanding or nonshrinking, regulated-set, and air-entraining cements. In the U. S. they are also classified as Types I, IA, II, IIA, III, IIIA, IV, and V,[3] the distinctions being made on composition. Types I, II, and V are commonly used in pavement concrete.

The specific gravity of the particles of Types I through V portland cement generally ranges from about 3.12 to 3.17. Other portland cements such as expansive cements and regulated-set cements may have specific gravities as low as 3.06 and 2.99, respectively. The unit weight of bulk cement varies with degree of compactness with one cubic foot of bulk cement generally being assumed to weigh 94 lb. The particles of cement range in size from about 0.03 micron (0.000001 in.) in diameter to 80 microns (0.0032 in.).[2] The major portion of cement will pass a No. 200 sieve (74 micron opening). The rate of hardening and other properties are controlled to a considerable degree by the relative proportions of the essential compounds and by the fineness to which the cement is ground.

7.2.3 Aggregates. Many different types. shapes, sizes, and qualities of stone have been used as aggregate in portland cement concrete pavements. Typical aggregates include natural sand, manufactured sand, gravel, crushed gravel, crushed stone, and air-cooled iron blast-furnace slag. Any aggregates used should conform to the quality requirements of "Specifications of Concrete Aggregates" (ASTM C33).* Gradation limits for any given project are usually stipulated in the job specifications. Aggregates should be resistant to weathering action and be of such composition that no unfavorable reaction takes place between the aggregate minerals and the components of the cement. The aggregate should also not contain any impurities which would affect the strength and soundness of the cement paste.

7.2.4 Admixtures. Admixtures are used to modify the properties of concrete so it will be more suitable for a particular purpose. Their use to obtain desirable characteristics should be based on appropriate evaluation of their effects upon specific combinations of materials and upon economic considerations. Certain admixtures can improve workability and permit easier placement. Others can accelerate the set, permitting earlier finishing, removal of forms, and opening of lanes to traffic, as well as reduce the time of protection against freezing during cold weather. Others can retard the set of concrete where rapid setting is undesirable. Many retarders accelerate strength gain once initial set is attained. Detailed descriptions of admixtures and their use is contained in References 4, 5, and 6.

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*American Society for Testing of Materials Standard C33.

7.3 NATURE OF STRENGTH

Most general discussions of portland cement concrete properties are prefaced with some comments on the "nature of strength" because there is not a complete understanding of the exact mechanisms of resistance that occurs in concrete under load. This lack of understanding occurs because concrete is not a simple solid but is a heterogenous mixture of solids and cement gel. Neville[7] defines this solid portion as being aggregates, remnants of unhydrated cement, and calcium hydroxide. Numerous failure theories have been proposed for concrete with many of them contained in references 8 to 20. In general, it is believed that the failure of concrete under load takes place through progressive internal cracking caused by the formation, under increasing load, of numerous microcracks and, that one crack, either initiated singly or developing from the joining of many smaller cracks, extends through the concrete thus causing separation of the concrete and failure. The ability of the concrete to resist this cracking must, of course, be found in the specific materials which compose the concrete and the manner in which they act together.

From a mechanical point of view concrete is not one, but many different materials which behave differently depending on the strength of the components, the strength of the bond between components, the ratio between the modulus of elasticity of the components, and the fractional volumes of the components. In addition, concrete strength also depends on the size, shape, and quality of the test specimens, friction of the testing machine platens, rate of loading, stiffness of the testing machine, curing conditions, and many other factors. Moreover, there also exists a difference between strength of concrete measured on test specimens and the strength of the same concrete in a structure.

7.4 TYPES OF STRENGTH

With respect to strength, concrete is usually characterized by compressive, tensile, flexural, shear, and bond (with reinforcement) strengths. Except for pavements, most concrete structures are designed to resist compressive stresses but not tensile stresses. Hence, in most instances, compressive strength is the criterion of quality. For pavements, flexural strength (Modulus of Rupture) is usually the criterion of quality. The following discussion will concern itself only with compressive, tensile, and flexural strength as these are more commonly stated in pavement work.

7.4.1 Compressive Strength.

The compressive strength of concrete is usually determined by loading a cylindrical specimen of height equal to twice the diameter. This specimen is usually moist cured at 70± 5 F for 28 days and then subjected to a slow or "static" loading at a specified rate until rupture occurs. The loading is usually accomplished in 2 to 3 minutes. Values of strength obtained in this manner range from 2000 to 9000 psi, the most common values being of the order of 3000 to 4000 psi. Compressive strength requirements for pavements vary depending on type of pavement, pavement thickness, subbase strengths, anticipated pavement loads, and other considerations. American Concrete Institute Committee 316 on Construction of Concrete Pavements and Concrete Bases has recommended that specified flexural and compressive concrete strengths for design and durability purposes should not be less than 650 psi and 4000 psi, respectively, at 28 days age.[21] Mather[22] noted as early as 1962 that concrete compressive strengths as great as 18,000 psi had been produced. Saucier, et al,[23,24] showed that concrete with compressive strengths of 10,000 psi or greater at 90 days age could be routinely produced by using certain precautions in the field. It was later shown[25] that by using water-cement ratios of less than 0.26 and proper proportioning, that workable concretes having 90-day strengths of 15,000 psi could be produced.

7.4.2 Tensile Strength.

Although considerable attention has been paid to the investigation of the "tensile" strength of concrete in recent years, there is general disagreement among investigators as to which test of concrete provides a "true" measure of the tensile strength. The type of tests used can be categorically described as either "direct" or "indirect" methods for obtaining a numerical representation of tensile strength, The direct tensile strength tests usually involve long, axially loaded specimens which are pulled in a testing machine until fracture occurs. The direct tensile stress is simply the load divided by the cross sectional area of the specimen at the point of failure. If the loading on the specimen is perfectly uniaxial (a possible but highly unlikely situation for most testing), the actual tensile stress is determined. The reproducibility of this test depends to a large extent upon the type of method used and to what degree the extraneous stresses caused by specimen gripping are eliminated.

A number of indirect methods for obtaining a measure of the tensile strength of the concrete have also been developed in hopes of providing a simpler or more reproducible, or both, test procedure than the direct tension test. These methods are:

a. Flexure test.
b. Splitting tension test.
c. Torsion tension test.
d. Ring tension test.

The flexure test is discussed in Section 7.4.3.

The splitting tension test as it is commonly used in the United States involves the placing of a cylindrical specimen horizontally between the loading surfaces of a testing machine so that the load is applied to the specimen along its entire length on two diametrically opposed lines. The application of a compressive force along these two opposite lines produces a biaxial stress distribution within the specimen. Immediately adjacent to the lines of load application, regions of high compressive stresses are developed. These stresses vary to a minimum at the center of the cylinder and are accompanied by an almost constant tensile stress over approximately three-quarters of the vertical plane between the two lines of load application. The concrete, being considerably weaker in tension than in compression, fails along this vertical plane because of the tensile forces developed there. The magnitude of the average tensile stresses along this plane at the time of failure is considered to be the tensile strength of the concrete.

The torsion tension test uses long cylindrical specimens which are subjected to pure torsion by means of a specially designed loading frame. The state of pure shear being developed at a point in a specimen being subjected to torsional loading is also accompanied by normal stresses (tensile and compressive) at the point on planes that bisect the angles between the planes on which the shearing stresses act. The magnitude of these normal stresses is equal to those of the shearing stresses, and if the specimen is made of a brittle material (such as concrete) which is weak in tension, the diagonal tensile stresses will reach a value that will cause the specimen to fracture. This value is then considered the tensile strength of the material.

The determination of the ring tensile strength of rock-like materials and concrete has been approached using two different methods. The first method uses discs with concentric holes (e.g., rings) in a diametral compression test identical to the cylinder splitting test described earlier. The basic idea in this test is to change, by addition of the hole in the disc, the rather uniform tensile stress field which occurs across approximately the center three-fourths of the failure plane such that the tensile stress component at the edge of the hole is increased. In doing this, it insures that the origin of the fracture is known. The only component of stress acting at the edge of the hole is the unidirectional tensile stress which then would be the stress which causes fracture and, hence, represents the tensile strength of the concrete.

The second method used for the ring tension test utilizes a uniform hydrostatic pressure which is applied radially against the inside periphery of the ring. The application of this pressure produces tangential tensile stresses and radial compressive stresses throughout the entire volume of the ring with a uniformly distributed maximum tensile stress occurring along the entire internal periphery of the ring. The magnitude of the radial compressive strength is quite small when compared with the tangential tensile stress so that when failure occurs, it is the reuslt of the tensile stresses. If the ratio of the radius of the ring to its wall thickness is less than 10 (as is generally the case for tests of concrete), the classic equations for determining stresses in thick-walled cylinders due to internal hydrostatic pressure can be used to determine the tangential tensile stress at failure.

All of the stress analysis techniques used to provide a measure of tensile strength from the output generated by using one of the above indirect test methods and the flexural test make the assumption that the concrete obeys Hooke's law of linear stress-strain proportionality whereas in fact the stress-strain relation for concrete is curvilinear almost from the onset of loading. This assumption then introduces an error into the tensile strength determination and the actual numerical representations of tensile strength obtained for any of the indirect test methods does not represent the "true" tensile strength of the concrete. On the other hand, the ultimate tensile strength obtained from direct tension testing is independent of the stress-strain relationship in the concrete and is simply the load at failure divided by the cross-sectional area at failure. Unfortunately, the direct tension test is complicated by difficulties in mounting the specimens and uncertainties as to secondary stresses induced by the clamping or holding devices and hence is never used for purposes of concrete control.

In using values for tensile strength, the user should be aware of how the value was obtained and how it relates to other values of tensile strength. In general, flexural tests give the highest tensile strength value for a given mixture followed in decreasing order by ring tensile strength, tensile splitting strength, and direct tensile strength.[26] The position in this ranking of tensile strength from a torsion test is not known. The ratio of tensile strength to compressive strength decreases with increasing compressive strength. Hoff[26] found that for 1-in. maximum size aggregate used in concrete over the compressive strength range of 3000 to 9000 psi, the following ranges of tensile/compressive strength ratios expressed as percent existed:

Type of Test	Size of Specimen	Range of Strength Ratios, %
Flexural	6-in. x 6-in. x 30-in. beam	20 to 11
Ring	12-in. rings, 3-in.-thick walls	16 to 9
Splitting	6-in. x 12-in. cylinder	13 to 8

In general, the tensile strength of concrete is usually assumed to be only 10 percent of its compressive strength.[27] Many investigators believe that there is an upper limit to tensile strength regardless of the compressive strength because of the limiting values of bond between aggregate and paste. This upper value is believed to be approximately 600 psi[27] although higher values have been reported.[23]

7.4.3 Flexural Strength. The flexure test uses a plain concrete or mortar beam of a given span length, width, and depth, which is supported at its ends and loaded at either its third points or at its center. The flexural strength of the beam is represented by a "modulus of rupture" which is defined as the tensile stress in the outer fiber of the beam at failure of the beam. This tensile stress is developed by the beam action and determination of the modulus of rupture assumes that the stress and deformation in the beam are directly proportional to the distance from the neutral axis of the beam. The results of the flexure test are dependent on where the beam is loaded (third-point or center loading), the rate at which it is loaded, the size and configuration of the beam, and on the residual stresses in the beam caused by moisture variations within the beam.

As noted in Section 7.4.2, the flexural test usually gives the greater tensile value of all the indirect tensile tests, ranging from approximately 20 percent of the compressive strength at lower concrete strengths (3000 psi) to 11 percent at higher concrete strengths (9000 psi). It ranges from 60 to 100 percent higher than the direct tensile strength of concrete.[28] In general, the flexural strength is assumed to be 15 percent of the compressive strength.[27] As in the case of tensile strength, it can have a limiting upper value depending on the surface condition and strength of the aggregate used. Rough-textured or angular aggregates result in relatively high flexural strengths as compared to smooth glassy aggregate used in the same concrete. Saucier, et al,[18] reported flexural strengths of 1050 psi for concrete having a compressive strength of 15,000 psi.

7.5 FACTORS AFFECTING STRENGTH

Many different things can affect the strength of concrete. Generally, if a particular situation or phenomena affects one type of concrete strength, such as compressive strength, it also will affect the other strength characteristics of the concrete, although perhaps not in the same proportion. The following sections point out a few of the major areas that affect strength. The effects on strength of freezing and thawing, reactive aggregates, corrosion of embedded items, and chemical effects are described in Section 7.8.

7.5.1 Component Materials. The component materials of portland cement concrete are the cement, aggregate, water, and admixtures (if used).

The effect of the cement constituent is manifested through differences in chemical composition and fineness. The chemical composition is a combination of lime (CaO), silica (SiO_2), alumina (Al_2O_3), magnesia (MgO), sulfur anhydride (SO_3), and iron oxide (Fe_2O_3). When these components are intimately mixed and burned to equilibrium, the following compounds are formed:

Compound	Chemical Description	Cement Chemist Notation
Tricalcium silicate	$3CaO \cdot SiO_2$	C_3S
Dicalcium silicate	$2CaO \cdot SiO_2$	C_2S
Tricalcium aluminate	$3CaO \cdot Al_2O_3$	C_3A
Tetracalcium aluminoferrite	$4CaO \cdot Al_2O_3 \cdot FeO_3$	C_4AF
Uncombined magnesia	MgO	---
Uncombined lime	CaO	---
Calcium sulfate	$CaSO_4$	---

The calcium silicate compounds are generally believed to be the ones most effective in developing strength. Cements relatively high in C_3S (alite) gain strength more rapidly and produce more heat per unit of cement during hydration. Both of these characteristics are further enhanced when there are larger amounts of C_3A (alumina) in the cement. These larger amounts of C_3A, although increasing early strengths, cause lower strengths at later ages. Cements relatively high in C_2S (belite) hydrate more slowly with accompanying lower strength and lower heat of hydration.

Cements with C_3A contents above 5 percent have an increasing susceptibility to sulfate attack with subsequent loss in strength. C_4AF is not a significant contributor to strength but when present in large amounts, may cause slight decreases in strength. Excessive amounts of MgO and free CaO may, under certain conditions, participate in reactions which produce strength-reducing deleterious expansions.

Limiting values of these various compounds are contained in ASTM C 150, Standard Specification for Portland Cement.[3] Some typical compound compositions for the five principal types of portland cement are contained in Table 7.1.

Normal portland cements generally have a specific surface (fineness as determined by air permeability method) of about 3000 to 4000 sq cm/gm. High early-strength cements may have specific surfaces of 5000 sq cm/gm or greater. In general, the higher the fineness the greater the rate of hydration thus causing greater early strength and a more rapid generation of heat. The total amount of influence of fineness on strength will vary, however, with the cement and the concrete mixture.[2] Lea[2] has noted that an increase in specific surface from 3200 to 4500 sq cm/gm (by air permeability method) raises the compressive strength at 1 day by 50 to 100 percent, at 3 days by 30 to 60 percent, and at 7 days by 15 to 40 percent. Increases in fineness beyond about 5000 sq cm/gm (air permeability) only produce a relatively small increase in strength except at less than 1-days age. Brunauer, et al,[29,30] however, found that by grinding cements as fine as 8000 sq cm/gm (air permeability), excluding all gypsum, and reducing the water-cement ratio to 0.2, that pastes of very low shrinkage and having 90-day strengths in excess of 30,000 psi could be satisfactorily made. The elimination of the gypsum results in a paste containing no sulfo-aluminates or sulfo-ferrites which are considered as being responsible for a considerable part of the volume changes of normal cement.

Aggregates can affect the strength of concrete by their grading, surface texture, shape, strength, and stiffness (elasticity). Changes in sand grading over an extreme range have no material effect on compressive strength of mortar and concrete specimens when water-cement ratio and slump are held constant.[31] These changes under the conditions mentioned do cause the cement content to change, however. Changes in coarse aggregate grading always result in changes in the proportions of a mixture in which it is used in order to maintain a desired level of workability. It is these proportioning changes which, for the most part, affect the strength. The amount of cement required to produce maximum compressive strength at a given age with a given aggregate will vary with the maximum size of aggregate involved. Greater strengths can be obtained at higher cement contents for all sizes of aggregates until a maximum strength is reached beyond which the addition of cement produces no increase in strength.[31] The compressive strength at which the addition of cement produces no further increase in strength is higher for the smaller size aggregates than for the larger size aggregates. In lean mixtures, large aggregate gives highest strengths while in rich mixtures it is the smaller aggregate size that yields highest strength. In other words, peak compressive strengths are reached at progressively larger aggregate sizes as the cement content decreases. Standard gradation curves or tables are available for optimizing concrete strengths.

Under compression loading, the stress at which cracks begin to form in concrete depends largely on the properties of the coarse aggregate. Smooth gravel leads to cracking at lower stresses than does rough and angular crushed rock, probably because mechanical bond between the paste and aggregate is affected by the surface properties and, to a certain degree by the shape of the coarse aggregate.[32] The properties of aggregate thus affect the cracking load (as distinct from the ultimate load) in the same manner for both compressive and flexural strength, thus making the relationship between the two independent of the type of aggregate used. The flexural and compressive strengths of concrete do depend, however, on the type of coarse aggregate used since (except in high-strength concrete) the properties of aggregate, especially surface texture, affect the ultimate strength in compression very much less than the strength in tension.[33]

The influence of the type of coarse aggregate on the strength of concrete varies in magnitude and depends on the water-cement ratio of the mixture. For water-cement ratios below 0.4 the use of crushed aggregate has resulted in strengths up to 38 percent higher than when gravel is used. With an increase in the water-cement ratio the influence of aggregate falls off, presumably because the strength of the paste itself becomes paramount, and at a water-cement ratio of 0.65 no difference in the strengths of concretes made with crushed rock and gravel has been observed.[33]

Kaplan[34] observed that the flexural strength of concrete is generally lower than the flexural strength of corresponding mortar. Mortar appears to be the upper limit to the flexural strength of concrete with additions of coarse aggregate generally reducing the strength. On the other hand, the compressive strength of concrete is higher than that of mortar. Kaplan[34] attributes this to the mechanical interlocking of the coarse aggregates when subject to compression. Neville[31] has noted, however, that this behavior does not apply generally and is a much more complex problem than stated by Kaplan.

Aggregates should not contain contaminating substances or be unsound. Aggregate is commonly contaminated by silt, clay, mica, coal, humus, wood fragments, other organic matter, chemical salts, and surface coatings and encrustations. Such contaminating substances in concrete act in a variety of ways to cause unsoundness, decreased strength and durability, and unsightly appearance. Mineral or rock particles that are physically weak, extremely absorptive, easily cleavable, or swell when saturated are susceptible to breakdown through exposure to natural weathering processes. The use of such materials in concrete will reduce strength. A more detailed description of the phenomena involved with these problems can be found in Section 7.8.

Usually any potable water is suitable for use as mixing water for concrete. There have been exceptions to this, however. Mixing water containing considerable amounts of soluble sulfate may result in a delayed reduction of compressive strength of the concrete. Mixing water containing 1.0 percent (10,000 ppm) sulfate produced no significant reductions in strength at 28-days age but at one-year age, the same concretes made with Types I and V cement showed strength reductions of 15 and 10 percent, respectively, from the strengths of similar concrete made with tap water.[31] Concrete made with sodium chloride (common salt) solutions showed significant reductions in strength at ages greater than 7 days for all concentrations. Five percent of ordinary salt reduced the strength about 30 percent. A highly carbonated mineral water containing only small quantities of sulfates and chlorides gave a strength ratio as low as 80 percent.[31] Further discussions of the effects of water quality are contained in Section 7.8.4.4.

The early strength of concrete can be improved by the inclusion of accelerating admixtures. Common accelerators are calcium chloride and sodium and aluminum-based commercial products. Their use requires special precautions in handling and placing of the concrete to avoid delays as other characteristics such as slump loss and stiffening are also accelerated. Limits for total amounts of accelerators to be used are usually provided by the manufacturer. Excessive amounts may adversely affect the performance of the concrete. Their use in controlled amounts may still adversely affect other characteristics of the concrete over the long run. Air-entraining admixtures, by virtue of providing a controllable void system in the concrete, tend to reduce concrete strengths. For the usual range of entrained air contents (3 to 8 percent), a rule-of-thumb of a 5 percent strength reduction for each percent entrained air is commonly used. Water-reducing admixtures generally improve the compressive strength of concrete by allowing water reductions of up to 25 percent of the total mixture water while maintaining the same level of workability. This results in a lower water-cement ratio for the concrete. The Bureau of Reclamation reported[31] that concrete containing recommended dosages of water-reducing admixtures had compressive strengths 18 percent greater than concrete which did not contain the agents. Over-dosages of the water-reducing admixture can negate any benefits, however. Some water-reducing agents have, however, adversely affected the concrete durability over extended periods of exposure.

7.5.2 Mixture Proportions. For given materials, the strength of concrete at a given age and cured under prescribed conditions is generally assumed to depend primarily on the water-cement ratio and the degree of compaction of the concrete. In practice, the water-cement ratio is the largest single factor in the strength of fully compacted concrete.

The influence of the water-cement ratio on strength is due to the fact that the water-cement ratio determines the porosity of the hardened cement paste at a given stage of hydration. Neville[7] has noted that if the water/cement ratio is higher than about 0.38 (by weight) all the cement can hydrate, but capillary pores will also be present in the cement paste. Some of the capillaries will contain excess water from the mixture; others will fill by imbibing water from outside curing sources. Other investigators have reported[35] that all the hydration will occur when the water-cement ratio is as low as 0.20 (by weight) and that any water in excess of this amount can be considered as pore space.[36] It is this pore space which reduces the amount of strength which the paste can develop. It has been speculated that the zero-porosity strength of fully hydrated cement paste, if such a paste could be made, would be between 40,000 and 60,000 psi.

From practical considerations principally related to needed workability of the mixture, the water-cement ratios of good structural concrete are generally in the range of 0.35 to 0.45 (by weight). With the use of newly developed water-reducing admixtures (Section 15.15.12.2), water-cement ratios considerably below 0.35 may be possible. The relationship between strength and water-cement ratio is generally depicted in graphical form as a hyperbola. It is sometimes shown in the form of a strength versus cement-water ratio which plots approximately linear between water-cement ratios of 0.4 to 1.1. This makes interpolation much easier. Any of these relations apply only to a given cement and aggregate and changes in these materials will cause shifts in the relationships. The range of validity of the water-cement ratio rule is somewhat limited, however.[7] At very low water-cement ratios, full compaction is usually no longer possible and additional voids begin to occur in the system, thus adversely affecting strength. It has also been observed[33] that mixtures with a very low water-cement ratio and extremely high cement contents (800 to 900 lb per cu yd of concrete) exhibit retrogression of strength, especially when large size aggregate is used. This behavior has been attributed to shrinkage of the paste at later ages, which reduces cement-aggregate bond and causes cracking because of aggregate restraint.

The proportioning effects of aggregates are discussed in Section 7.5.1.

7.5.3 Curing Conditions. For satisfactory development of strength in concrete, it is not necessary for all the cement to hydrate, and in most cases, not all of it does. To achieve the maximum strengths possible for a given combination of materials it is necessary that as much hydration as possible does take place, however. This is usually accomplished by having long periods of moist storage or curing, especially at early ages of the concrete. More than enough water is usually mixed in the concrete for complete hydration of the cement to occur. The purpose of applying moisture to the concrete is to prevent the loss of this moisture from the concrete. This may also be accomplished by sealing the concrete with a membrane to reduce moisture losses.

The temperature of the concrete during the early age of the concrete also has an important bearing on the strength of the concrete. When concrete is cast and maintained at a given constant temperature, the higher the temperature (within limits), the more rapid the hydration and resulting gain in strength at ages up to 28 days. A comparison of some typical strengths for concrete cured at various temperatures is shown in Table 7.2. It should be noted that concrete cast at low temperatures (40 F) and stored at low temperatures for a month or more before being stored at higher temperatures will, from the age of 3 months on, be stronger than similar concrete stored continuously at the higher temperature. Care should be taken to prevent freshly placed concrete from freezing, however. If fresh concrete is frozen while there is considerable free water in the mixture, and the paste, as it goes from its liquid to solid phase, has not attained sufficient strength to resist the expansive forces associated with an increase in volume of the water, the internal structure of the concrete can be disrupted with corresponding decreases in strength. The 28-day compressive strength of concrete which has been frozen may be reduced 30 to 50 percent of the strength of concrete which has not been frozen.[37] When concrete is cast and maintained at a given temperature for several hours and then cured at 70 F, the higher the initial temperature (within limits) the lower the 28-day strength, although differences may not be large. When concrete is cast and maintained at 70 F for several hours and then cured at various constant temperatures, the lower the curing temperature (within limits), the lower the strength at ages up to 28 days.[38]

It can be generalized that if the curing temperature is higher than the initial temperature of casting, the resulting 28-day strength will be higher than that for a curing temperature equal to, or lower than, the initial temperature. This may not necessarily hold true at later ages, however.

7.5.4 Changes with Time. As can be seen from the comments in Section 7.5.3, the development of strength in concrete is both a function of its curing conditions and time. Under ideal curing, the strength development goes on for many years. Long-term tests have shown that strength development can still be active after 50 years, although the strength increases may be relatively small.[39,40] Washa and Wendt[40] noted that comparable concrete cylinders made with cements having a relatively high C_2S content and a low surface area that are stored outdoors generally increased their compressive strength as the logarithm of the age out to about 50 years. Concretes made with finer cements having a relatively lower C_2S content appeared to reach their peak strength between 10 and 25 years and showed strength regression thereafter. Their data indicated compressive strengths between 2.0 and 2.5 times the 28-day strength were obtained at 50 years age. Some design procedures allow for additional strength gains of the concrete when the full design load is applied at a minimum age of the member.[41] An age factor of 1.24 times the one-month strength is used for low-strength concrete when the member is loaded at 12-months age. For better grades of concrete an additional strength of 1450 psi is allowed instead of applying an age factor.[41]

Most expressions for relating strength to time are derived for a particular type of cement. As most cements gain strength at different rates, the prediction of strength should be based on experimental results for the cement in question.

The temperature during curing also controls the rate of progress of the reactions of hydration and consequently affects the development of strength of concrete with time. The relationship between time, temperature, and strength is often expressed as a summation function (called maturity) of the product of time and temperature, referenced to an origin found experimentally to be 11 F.[42] That is,

$$\text{Maturity} = \sum \left[\text{time x (Temperature - 11)}\right] \qquad (7.1)$$

Below 11 F concrete does not appear to gain strength.

Maturity is expressed in degree F-hours or degree F-days. Plots of strength versus the logarithm of maturity usually produce a straight-line relationship. From this it is possible to express strength at any maturity as a percentage of strength at any other maturity, thus allowing strength predictions for any combination of time or age and curing temperature condition. The maturity rule applies fairly well when the initial temperature of concrete is within normal concrete curing temperature regimes and there is no loss of moisture by drying during the period considered.

7.5.5 Size and Shape of Specimens. The same concrete used in a variety of different sized or shaped specimens will produce different concrete strengths because of the influence of the size and shape of the specimen and, to a lesser degree, of the possibility of a better compaction and a larger water gain during moist curing in smaller specimens. When a given concrete is tested in compression by means of cylinders of like shape but of different sizes, the larger the specimen, the lower the strength. The same behavior generally occurs for prism-shaped flexural test specimens. The explanation usually offered to explain size effect is the probability of the presence of a weak element. It is assumed that the larger the volume of the concrete subjected to a given level of stress, the more likely the volume will contain an element of a given extreme low strength or a flaw. Table 7.3 has been adopted from a curve used by the Bureau of Reclamation[31] to give the relationship between strength changes and test sylinder size and shows some typical strength changes that might be expected. Neville[43] has suggested the following general relation between the strength of concrete and the size of the specimen:

$$\frac{P}{P_6} = 0.56 + 0.697\left(\frac{d}{\frac{V}{6h} + h}\right) \qquad (7.2)$$

where

P = concrete strength

P_6 = strength of a 6-in. cube

V = volume of specimen

h = height of specimen

d = least lateral dimension of the specimen

It should be noted that the size effect disappears beyond a certain size so that a further increase in size does not lead to a decrease in strength.[31,33] Also, decreases in strength with increases in specimen size are less pronounced in lean mixtures than in rich mixtures.

The greater the ratio of specimen height to diameter, the lower the strength indicated by the compression test. Table 7.4 gives some typical strength correction factors for cylindrical specimens whose height to diameter ratios are less than 2, which is the preferred ratio for standard test cylinders. The product of these correction factors and the observed strength of the actual cylinders tested provides indicated strengths comparable to those that would have been obtained from a standard specimen. For ratios of 1.5 or greater, small variations in height of specimen do not greatly affect strength.

7.5.6 Types of Loading. Concrete, when subjected to different types and rates of loading, will produce values of apparent strength which may vary considerably. In general, the lower the rate at which stress increases, the lower the recorded strength. This is probably due to the increase in strain with time owing to creep (See Section 7.6.7), and, when limiting strain is reached, failure takes place largely independently of the value of stress applied.

Within the range of customary testing procedures, the effect of rate of load is not large. With ordinary laboratory machines, the practical range of speed of loading is between 10 and 100 psi/sec, and within this range the recorded strength varies only between 97 and 103 percent of the strength obtained at a load rate of 30 psi/sec. Above load rates of 10,000 psi/sec, the dynamic effects due to inertia become significant and the apparent strengths increase significantly (See Section 7.10).[31,45,46] Early tests at the University of Illinois[47] indicated that the relation between strength and rate of loading was approximately lgarithmic in the form

$$S = S_1 (1 + K \log R) \tag{7.3}$$

where

S = strength at a given rate of loading R psi/sec
S_1 = strength at a rate of 1 psi/sec
K = a constant (approx 0.08 for 28-day tests)

This relationship is still valid for concrete tested in compression. Flexural tests are also affected by the speed of loading in the same manner as compression tests and follow the same general relationship noted in Equation 7.3. The value of the constant K will be different, however.

7.6 DIMENSIONAL CHANGES

Portland cement concrete can undergo dimensional changes for numerous reasons. When under short-term load, it can change dimensions due to the mechanical forces acting on it. Under long-term sustained loads, it may undergo time-dependent deformation called creep. In an unhardened state, plastic shrinkage can occur. Once the concrete has hardened, it may undergo dimensional changes due to autogenous volume changes, swelling, drying shrinkage, carbonation shrinkage, and expansion due to deleterious substances including aggregates or chemical attack. Dimensional changes also occur from simple temperature changes. The following paragraphs highlight some of these phenomena. The effects of deleterious substances and chemical attack are found in Section 7.8 on Durability.

7.6.1 Plastic Shrinkage. Plastic shrinkage is a volume change which takes place while the concrete is still in its unhardened state. It can lead to cracking problems which can contribute to the further deterioration of a concrete pavement at later ages.

During the hydration of the cement phase of the concrete, the volume of the paste fraction decreases while the paste is unhardened. Normally this volumetric contraction is of the order of 1 percent of the absolute volume of dry cement.[7] If there is a loss of water by evaporation from the surface of the concrete or by suction of dry concrete below the surface, the volume change of the unhardened paste is even greater. Whether this change in volume is the sole cause of shrinkage cracking is not clearly defined. Cracks have occurred when no evaporation or suction occurred. It has been suggested that plastic shrinkage cracking is associated with differential settlement of fresh concrete due to some obstruction to uniform settlement, such as coarse aggregate or reinforcement. Both of these conditions can be eliminated by proper porportioning of the mixture and the provision of adequate moist curing, applied at the proper time, to minimize evaporation.

7.6.2 Autogenous Volume Change. This type of volume change does occur in large sections of mass concrete but can be ignored for pavements. It is noted here only for information purposes.

When a section of concrete has a supply of water available to it, hydration of the cement will continue and lead to a slight swelling of the concrete. If no moisture movement to or from the concrete is allowed and temperature remains constant, hydration will still continue for some time with the moisture available within the concrete. The resulting self-desiccation will produce an autogenous shrinkage, however. Usually this shrinkage is assumed to be a portion of the total drying shrinkage (Section 7.6.4).

7.6.3 Swelling. Cement paste or concrete continuously cured in water from the time of casting exhibits a net increase in volume and an increase in weight due to the adsorption of water by the cement gel. Linear expansions of 100 to 150 microstrains after one year's water curing have been observed for concrete made with 500 lb of cement per cubic yard of concrete.[48] As in the case of autogenous volume changes, this type of behavior is usually not a consideration for volume changes in pavements because they are never continuously cured in water over their operational life.

7.6.4 Drying Shrinkage. The drying shrinkage of concrete is caused by the withdrawal of water from the concrete and is usually the result of the presence of the concrete in an unsaturated air environment. The water is lost from the cement paste in the concrete and a part of this loss is irreversible. The change in volume of drying concrete is less than the volume of water lost during drying, however. The loss of free water, which occurs first, causes little or no shrinkage. As drying continues, adsorbed water is removed and this results in a linear change in dimensions of the cement paste.

The drying shrinkage of concrete is influenced by many factors. The most important is the aggregate which restrains the amount of shrinkage which can occur. Pickett[49] reported that the shrinkage of concrete, S_c, to shrinkage of neat paste, Sp, depends on the aggregate content, g, in the concrete and is:

$$S_c = S_p(1-g)^n \tag{7.4}$$

where n takes into account the aggregate content of the mixture and allows for unhydrated cement.[50] Values of 1.2 to 1.7 have been reported for n.[48] The size and grading of aggregate does not by itself influence the magnitude of shrinkage but they do influence the proportions of the concrete which in turn influences the shrinkage. The elastic properties of the aggregate determine the degree of restraint offered, with aggregates of high elasticity offering more restraint than aggregates of low elasticity.

Within the range of ordinary aggregates, there is considerable variation in shrinkage of concrete. In general, shrinkage varies inversely with the modulus of elasticity of concrete at early ages.[51]

The water content of concrete affects shrinkage insofar as it reduces the volume of restraining aggregate. The water content of a mixture is an indicator of the order of shrinkage to be expected but is not a primary factor causing shrinkage.[7]

The properties of cement have little influence on the shrinkage of concrete. Air entrainment has been found to have no effect on shrinkage.[52] Additions of calcium chloride to concrete have been reported to increase shrinkage by amounts varying from 10 to 50 percent.[53]

Shrinkage takes place over long periods of time. Neville[7] reported the following shrinkage rates for concretes of varying proportions and curing conditions:

Time of Occurrance	Percent of 20-year Shrinkage
2 weeks	14 to 34
3 months	40 to 80
1 year	66 to 85

Prolonged moist curing delays the advent of shrinkage, but the effect of curing on the magnitude of shrinkage once it occurs is small. The curing is beneficial from the standpoint that by delaying the shrinkage, the concrete can develop sufficient strength to resist any cracking that may result from the shrinkage. When moisture loss takes place at the surface of a pavement, a moisture gradient is established in the pavement with a resulting differential in shrinkage within the pavement. This shrinkage is compensated for by strains due to internal stresses. If the stresses arise gradually they may be relieved by creep. If the stresses in the tensile zone of the pavement exceed the tensile strength of the concrete, cracking in the surface will occur. Even if the curing has allowed sufficient strength to develop to resist these stresses, warping of thinner sections may occur if the drying takes place in an unsymmetrical manner.

Typical shrinkage values for concrete range up to 1500 microstrains with values of up to 4000 microstrains being reported for cement pastes. Enough is known about the causes of drying shrinkage that with the proper care and precautions, it can be controlled so as not to be a problem.

7.6.5 Carbonation Shrinkage. Carbonation shrinkage has been identified in concrete pavements. It is generally believed[2] that carbonation shrinkage is caused by the dissolving of crystals of $Ca(OH)_2$ in the cement paste in the presence of CO_2 in the atmosphere and moisture in the concrete to form $CaCO_3$ plus water. The $CaCO_3$ occupies less volume than the $Ca(OH)_2$, while the water is free to escape from the system. The result of both actions should cause shrinkage.

The rate of carbonation depends on moisture content of the concrete and the relative humidity of the ambient medium. The size of the concrete element is also a factor. Most occurrences of carbonation shrinkage have been attributed to the phenomena of drying shrinkage and have been included in the measurement of that phenomena. Carbonation of concrete does result, however, in increased strength and reduced permeability of the concrete because the water released by carbonation and the process of hydration and $CaCO_3$ reduces the voids within the cement paste. Both of these may improve durability.

7.6.6 Thermal Volume Changes. Concrete has a positive coefficient of thermal expansion. The value of this coefficient depends both on the composition of the mixture and on its hygral state at the time of the temperature change.

The two principal constituents of the concrete, the cement paste and the aggregate, have dissimilar thermal coefficients. These ingredients also occur in the mixture in varying proportions. This results in the concrete having a range of values for its thermal coefficient. In general, the coefficient of concrete is a function of the quantity of aggregate in the mixture and the coefficient of the aggregate itself.[54] Tables 7.5 and 7.6 show the influence of both aggregate content and type of aggregate on the coefficient of thermal expansion of concrete.

The significance of the differences between coefficients of aggregate and those of the cement paste is that if they differ too greatly, cement. Less hydration products are produced with a coarse cement at early ages than with a fine cement which has more surface area available to hydrate. The long-term porosity and permeability are unaffected by the type of cement used, however.

In general, the higher the strength of a paste, the lower its permeability because higher strengths are usually associated with higher cement contents and lower water-cement ratios. An exception to this is when drying shrinkage is present. This shrinkage may rupture some of the paste and thus open new passages to the infiltration of water.

Differences in permeability between the paste in a concrete and the concrete itself have been attributed to the presence of aggregate and degree of compaction of the concrete.[33] If the aggregate has a very low permeability, its presence reduces the effective area over which flow can take place. Furthermore, since the flow path has to circumvent the aggregate particles, the effective path becomes considerably longer so that the effect of aggregate in reducing the permeability of the concrete may be considerable.

Air entraining would be expected to increase the permeability of concrete. However, since air entraining reduces segregation and bleeding, and improves workability, and so permits the use of a lower water-cement ratio, the net effect of air entraining is not necessarily adverse.[7]

7.8 DURABILITY

The American Concrete Institute Committee 201 on Durability of Concrete defines durability of concrete as its resistance to deteriorating influences which may through inadvertence or ignorance reside in the concrete itself, or are inherent in the environment to which it is exposed.[61] The durability of concrete in various circumstances is determined very largely by the physical and chemical nature of its component parts. It can be affected by inferior materials, reactive aggregates, freezing and thawing, corrosion of imbedded items, chemical attack, biological attack, abrasion, erosion, cavitation, and temperature.

The use of suitable materials is of primary importance in producing durable concrete pavements. Specifications are available under which these materials can be obtained and there is little excuse for including materials of inferior quality. It is the most advantageous use of these materials to produce the finished product that poses the greatest problem to constructors, however. ACI Committee 201 has defined[61] a number of elementary factors which enter into the production of durable concrete. These are:

Water-cement ratio
Air entrainment
Proportioning concrete mixtures
Measuring, mixing, and placing
Finishing and curing

The water-cement ratio is a property of fresh concrete that has more to do with the quality of hardened concrete than any other because it is a measure of the quality of the hardened cement paste binder (See Section 7.5.2). Strength, impermeability, and most other sought-after characteristics in concrete are improved by lowering the water-cement ratio. The best concrete practice requires proportioning or combining the aggregate materials in such a manner as to permit the lowest water-cement ratio consistent with required placeability of the concrete. Air entrainment is also important as the introduction of a system of finely divided bubbles into the cement paste is essential for some exposures and in particular for freeze-thaw resistance.

Close attention to measuring, mixing, and placing is necessary to assure uniform quality of the concrete after it has been consolidated in the forms. Adequate curing is required to assure that the potentialities of the concrete as mixed may be realized in the completed pavement. Time, temperature, and moisture are the essentials.

Disintegration of concrete by weathering is caused mainly by the disruptive action of freezing and thawing and by expansion and contraction, under restraint, resulting from temperature variations and alternate wetting and drying. Concrete deterioration can also be attributed in whole or in part to chemical reactions between alkalies in cement and mineral constituents of concrete aggregates or to direct contact with certain chemical agents. The following paragraphs present an overview look at some of these phenomena.

7.8.1 Freezing and Thawing. Dry concrete is not damaged by freezing and thawing. Wet concrete exposed to repeated cycles of freezing and thawing is sometimes damaged or destroyed. The mechanism by which this occurs has been adequately described by Powers[62,63] and others. It is believed that the principle force responsible for freezing damage in ordinary concrete under usual winter exposures is the internal hydraulic pressure created by an expanding ice-water system during freezing.[64] This system of ice-water can occur in both the paste fraction and aggregate fraction of the concrete system.

When water in a saturated capillary pore in hardened cement paste freezes, the expansion produced in the ice-water system requires a dilation of the cavity of some 9 percent of the volume of frozen water, or the forcing of that volume of water out of the cavity into the surrounding paste, or a combination of both. If the cavity cannot accommodate this dilation or if the magnitude of the hydraulic pressure required to effect the transfer of the excess volume of water resulting from progressive freezing is too high with respect to the strength of the paste, the paste will be damaged and the pore size enlarged. Upon thawing and rewetting, the large pore fills with water and when freezing occurs again, the pore is enlarged further. The process continues until the concrete is destroyed.

The most apparent solution to this problem is to provide enough available space in the concrete to accommodate the additional volume created by freezing. This is done by providing "air entrainment" to the concrete. Experience has shown that it is necessary to have some empty space within about 0.005 in. of every point in the hardened paste for air entrainment to be effective.[31] Air entrainment is usually provided by adding to the concrete batch, at the time of mixing, a small amount (ordinarily less than 0.01 percent by weight of cement) of an air-entraining agent or admixture. Such agents are surface-active organic chemicals which have the property of stabilizing foams. These agents provide a large number of small bubbles rather than a smaller number of larger ones. Certain cements already have air-entraining agents in them and their use does not require the addition, at mixing, of the agent.

The aggregates in concrete frequently contain pore spaces which are capable of containing water which can freeze. The total amount of pore space, the size distribution of pores, and pore continuity are of prime importance in determining the durability of the aggregate in freezing and thawing.[31] In general, those rocks which, when used as aggreagte in concrete, do not have good freeze and thaw durability are those with moderate to high porosity, and which can attain and maintain a high degree of saturation when imbedded in concrete. All aggregates should be examined for freeze and thaw durability prior to their use in concrete where freezing and thawing may be a problem.

It has been generally concluded[61] that highly frost-resistant concrete can be produced by adhering to the following requirements:

a. Use of air entrainment.

b. Selection of aggregate with durability adequate for the exposure to be encountered.

c. Use of low water-cement ratio concrete properly handled, placed, and cured.

d. Design of the pavement to facilitate drainage.

e. Avoidance of materials or construction practices which may lead to other disintegrative processes.

7.8.2 Reactive Aggregates. It is generally believed that all aggregates are reactive, to some degree, when used in portland cement concrete. In most cases, it is not necessary to take into account any possible aggregate reactivity if aggregates of known good service record are used. Three principal reactions between aggregates have been identified. These are:

a. Alkali-silica reactions.

b. Cement-aggregate reactions.

c. Alkali-carbonate reactions.

In all cases, the reaction produces abnormal expansion with associated cracking and loss of strength.

Other damaging chemical reactions involving aggregates can also occur. These include the oxidation or hydration of certain unstable mineral oxides, sulfates, or sulfides that occur after the aggregate is incorporated in the concrete. Typical examples are the hydration of anhydrous magnesium oxide, calcium oxide, or calcium sulfate, or the oxidation of pyrite. Metallic iron may also occur in aggregates and be subsequently oxidized. Aggregates can also be contaminated by coal, gypsum, glass, and hard-burned lime or dolomite.[31] These are all deleteriously reactive substances. Coal often contains sulfur compounds, commonly iron sulfide, which oxidizes to iron sulfate and causes sulfate attack. Coal particles near the exposed surface of concrete may cause ugly brown stains. Gypsum is moderately soluble in water and gives rise to sulfate attack as described in Section 7.8.4.3. Most artificial glasses, as well as some natural ones, are expansively reactive with the alkalies from cement. Hard-burned lime and hard-burned dolomite incorporated with aggregate will react with water in the fresh concrete and carbon dioxide from the air and form hydroxides and carbonates with attendant swelling which can cause popouts.

7.8.2.1 Alkali-Silica Reactions. Deterioration of concrete in pavements and bridges due to the expansive reaction between constituents of some aggregates and alkalies from cements has occurred in numerous places in the U. S. and elsewhere. Typical manifestations of concrete deterioration from alkali-silica reaction are expansion, cracking, (usually map or pattern cracking), exudations of gel through pores or cracks which then forms jelly-like or hard beads on surfaces, reaction rims on affected aggregate particles within the concrete, and sometimes "popouts." It should be noted that these manifestations can also occur from other phenomena too, and that petrographic means must be used to determine the exact causes.

The rock materials which have been identified as potentially deleteriously reactive are:[65] opal; chalcedony; microcrystalline to cryptocrystalline quartz; crystalline quartz which is intensely fractured, strained, latitic, or andesitic glass, or cyptocrystalline devitrification products of these glasses. All these materials are highly siliceous. Some of the principal rocks which may contain the reactive materials mentioned are cherts, siliceous limestones and dolomite, rhyolites, dacites, andesites, siliceous shales, and phyllites. However, these rocks do not necessarily have to contain any of the reactive minerals.

The principal factors governing the extent of expansive reactivity of the aggregates are:

a. Nature, amount, and particle size of the reactive material.

b. The amounts of alkali and water available.

The most practical way to avoid expansion of concrete resulting from alkali-silica reaction is to not use reactive aggregates. Sometimes this is not economically feasible. When reactive aggregates must be used, it should be done only after thorough tests to determine the degree of reactivity of the aggregate have been conducted. Once this is known, appropriate limits on the alkali content of the cement can be established, or the use of an effective pozzolan can be recommended, or both, in order to reduce the potential for reaction.

7.8.2.2 Cement-Aggregate Reactions. Sand-gravel aggregates occurring along some river systems in the states of Kansas, Nebraska, Iowa, Missouri, and Wyoming have been involved in concrete deterioration that has been attributed to cement-aggregate reaction.[66,67] The aggregates from the various states are not similarly constituted and have various expansive tendencies. The principal manifestation of the expansion is map cracking. The underlying physical chemistry that occurs between the cement and aggregate to cause this is not fully understood, however. To avoid the problem, only aggregates with good service records should be used. Limiting the amount of alkalies in the cement with these aggregates has not always been a suitable remedial measure. One successful technique for avoiding the problem is to use 30 percent or more (by weight) of coarse limestone in combination with potentially deleterious cement-aggregate combinations. The reason this helps in reducing cracking is not fully understood, however.

7.8.2.3 Alkali-Carbonate Reactions. Certain dolomitic limestone aggregates found in a few places in the U. S. and Canada are suspected of being susceptible to this reaction. Many carbonate rocks are alkali-reactive but few are expansive. All of the expansively reactive carbonate rocks are generally thought to have the following features:[31]

a. They are dolomitic and contain appreciable quantities of calcite.

b. They contain clay.

c. They have an extremely fine-grained matrix.

d. They have a characteristic texture consisting of small isolated dolomite rhombs disseminated in a matrix of clay and finely divided calcite.

It is believed that the clay plays an important part in the mechanism of expansion, perhaps by imbibing water and swelling after being exposed in an active form during dedolomitization, or by acting as a semi-permeable membrane in the development of osmotic pressures, and perhaps by its presence as an interstitial material between the dolomite rhombs weakening the bonds between them.[68]

At present there are no well-established corrective measures that can be applied to deleterious carbonate aggregates to make them suitable for use in concrete. The use of low-alkali cement (0.1 to 0.2 percent equivalent Na_2O) has provided some relief but is not necessarily sufficient to prevent excessive expansion.[61] Dilution of the expansively reactive carbonate aggregate with nonexpansive coarse aggregate is found to reduce concrete expansion roughly in proportion to the amount of dilution. The reduction of maximum size of reactive coarse aggregate has also been found to reduce expansion. Pozzolan additions have not been effective in reducing expansions but they do decrease the initial rate of expansion.[31] In general, it is best to avoid the use of expansive carbonate rocks.

7.8.3 Corrosion of Imbedded Items.

7.8.3.1 Steel. Steel is used in concrete principally as reinforcement. Concrete ordinarily provides an almost ideal environment for protecting steel from corrosion. Its high alkalinity quickly causes the formation of a thin invisible protective oxide film on the steel. The low permeability of concrete minimizes penetration by water and air. Corrosion of reinforcing steel is therefore absent in the great majority of concrete structures but it can and does occur occasionally.

The iron in steel originally existed in nature in the oxidized state and, when given the opportunity, will attempt to return to that state and in general, become iron oxide or rust. The rust which is formed occupies more than twice the volume of the iron it came from and in the process of its formation can exert a pressure greatly exceeding the ability of even the strongest concrete to resist.

A number of conditions under which concrete may not provide permanent protection to the reinforcing are as follows:

a. Presence of cracks extending from the surface of concrete to the steel.

b. Carbonation of the cement which in turn produces cracking and reduces the alkalinity of the concrete.

c. Deterioration of the concrete from causes such as alkali-aggregate reaction, sulfate attack, or freezing and thawing.

d. Electrolysis caused by the passage of direct electrical current (external source) through concrete or concrete reinforcement.

e. Development of corrosion cells caused by a flow of electric current generated within the concrete itself. This current flow can occur in various spots in reinforced concrete because of differences in moisture content, oxygen concentration, electrolyte concentration, and by contact of dissimilar metals. The steel usually forms an anode in the system and corrodes.

Serious corrosion of reinforcing steel is usually accompanied by cracking of the concrete in a direction parallel to that of the steel. In advanced cases, spalling or raveling to the plane of the reinforcement has resulted. In less advanced cases, rust stains may be observed in the pores of the concrete and may appear in small cracks at the surface. Corrosion of the steel is frequently laminar in structure if the concrete is cracked and generally dark gray in color adjacent to the steel as opposed to the reddish color of atmospheric rusting. Corrosion can produce deep pitting and severe loss of cross section in bars. The nature of the corrosion products may not, however, be completely indicative of the nature and cause of the corrosion.[61]

It is generally believed that the permeability of concrete is one of the major factors affecting the process of corrosion of reinforcing steel. Concrete of low permeability contains less water under a given exposure and hence is more likely to have low electrical conductivity. It also resists the absorption of salts and their penetration to the reinforcing steel as well as providing a barrier against the ingress of oxygen.

Low water-cement ratios (0.45 or less) tend to produce less permeable concrete. Air entrainment is also recommended as an aid in securing better placement and increased resistance to water penetration. Permeability is also reduced by increased hydration of the cement and therefore adequate moist curing (14 days at normal temperatures) is essential. Good workmanship is also important to provide minimal segregation and adequate consolidation. Adequate concrete cover over the reinforcement to the top surface of a pavement or bridge deck is also essential. A minimum of 2 in. is often recommended.[31] Pavements and bridge decks should also have adequate drainage to prevent accumulations of water and de-icing salts.

Some steels such as stainless steels, chrome-nickel steels, and chromium-aluminum-silicon steels have been reported as having good resistance to corrosion in concrete.[69]

7.8.3.2 Other Metals. Other metals which are frequently imbedded in concrete are aluminum, lead, copper and copper alloys, zinc, iron, monel metal, stellite, silver, and tin. Dry concrete at normal temperatures is unlikely to promote corrosion of any of these metals.[31] Concrete is wet, however, whenever freshly mixed and placed and will contain a high level of free moisture and humidity for some time after hardening. It is possible for this free moisture to transport soluble chemical substances such as oxygen, calcium hydroxide, alkalies, and chlorides toward the metal, and any soluble corrosion products away from the metal. It also increases the electrical conductivity of the concrete, thus aiding any tendency for electrochemical corrosion. The moisture condition of concrete required to support active electrochemical corrosion of aluminum and other susceptible metals is not known with accuracy, but it is believed[31] that it could proceed, other conditions being favorable, at 90 percent relative humidity, and probably at 75 percent relative humidity, though at a lower rate.

Aluminum reacts in fresh concrete principally with alkali hydroxides from cement. One reaction product is hydrogen gas which can cause problems in unhardened concrete. Aluminum can also corrode with galvanic corrosion when it is coupled with steel and usually in the presence of calcium chloride. Lead is attacked by the calcium hydroxide in the concrete and becomes converted to lead oxide or a mixture of lead oxides. It too can suffer galvanic corrosion. Copper and its alloys are usually not corroded in concrete although the presence of soluble chlorides, very small amounts of ammonia, and nitrates have been reported as damaging or destroying the copper. The brasses and bronzes have been reported as having good corrosion resistance in concrete.[69] Zinc reacts chemically with alkaline materials but normally the reaction in concrete is superficial and may even help to promote bond between the zinc and the concrete. Zinc used in concrete containing calcium chloride has been reported as experiencing some chemical corrosion.

7.8.3.3 Plastics. The principal chemicals in concrete which could conceivably attack plastics are calcium hydroxide, sodium hydroxide, and potassium hydroxide. Few, if any, information has been published on their effects on concrete or vice versa. Plastic groups having excellent resistance to all three hydroxides are polyethylene, styrene copolymer rubber-resin blends, polyvinyl chlorides, and polytetrafluorethylene.[70]

7.8.3.4 Wood. The inclusion of wood in concrete, either in the form of sawdust, pulp, fibers, board, or timbers generally does not affect the concrete. An exception is in freshly-mixed concrete where the use of untreated sawdust, wood chips, or fibers will slow setting and reduce strengths because of interference with normal setting and hardening processes by the carbohydrates, tannins, and other substances in the wood. Concrete made with wood aggregate has considerably greater volume change on wetting and drying, or simply on change in external humidity, than concrete made with mineral aggregate. This may lead to cracking and warping in a pavement. On the other hand, the deterioration of timbers embedded in concrete has also occurred and has been attributed to the calcium hydroxide in the concrete which is said to cause dissolution of lipins and decomposition of pentosans, lignin, and cellulose.

7.8.3.5 Glass. Glass would not normally be expected to occur in a paving concrete, however, it may be introduced accidentally. The use of glass fibers in pavements in lieu of metallic reinforcing has been reported, however (See Chapter 9). Some glasses are expansively reactive with cement alkalies. The resulting expansion may cause severe damage to the glass or the concrete or both. Other glasses are nonexpansive in concrete. If glass is intentionally introduced into the concrete, only nonreactive glass should be used.

7.8.4 Chemical Effects. Lea[2] and Woods[71] have prepared excellent reviews on chemical effects on concrete. Much of the following information has come from these reviews.

7.8.4.1 Acids. Concrete is chemically basic, having a pH of about 13 and is therefore attacked by acids, which have pH values less than 7. Concrete is not resistant to strong solutions of sulfuric, sulfurous, hydrochloric, nitric, hydrobromic, or hydrofluoric acids, and will be destroyed by prolonged contact with any of these, though not necessarily at the same rate. Weaker solutions (less than 1 percent) will attack more slowly, but sometimes significantly. Woods[71] has noted that as a first approximation only, acidity represented by a pH of 5.5 to 6 may be considered the practical limit of tolerance of high quality concrete in contact with any of the acids named above, although he cautioned that pH value is not invariably a good criterion of the agressiveness of acids.

Organic acids may also exert deleterious effects on concrete.[2] They are generally divided into two groups: those having low molecular weights such as lactic, butyric, oxalic, and tartaric acids; and those having molecular weights such as oleic, steraric, and palmitic acids.

Lactic acid has a most destructive action on hardened concrete. This type of acid is common to the dairy and cheese industries. Concrete immersed in lactic acid solutions has its cement gradually dissolved away until only the aggregate remains. Butyric acid generally has the same effect but it is usually associated with sour silage storage.

Acetic acid also attacks hardened cement and concrete. A 5 percent solution in venegar will have a marked effect on concrete within a few months. Citric and malic acids have a similar action to acetic acid and in a 1 percent solution, will seriously attack portland cement concretes within a year or so. Tartaric acid is similar in its action on concrete to lactic and acetic acids. Oxalic acid on the other hand does have some slight effect on portland cement concretes, but its effects are not serious. It has in fact been used to treat concrete surfaces to form an insoluble film of calcium oxalate which in turn renders the concrete surface more resistant to other weak organic acids.

Organic acids of high molecular weight such as oleic, stearic, and palmitic acids together with the general series of unsaturated and saturated aliphatic acids all have a very definite action on concrete. These fatty acids of high molecular weight are usually encountered as constituents of fats and oils and will attack any unprotected concrete. The disintegration produced is usually more pronounced when the concrete is exposed to air, as in a floor or pavement, than when it is continuously immersed beneath a liquid.

7.8.4.2 Bases. For present purposes, bases may be defined as chemical compounds which yield hydroxyl (OH-) ions in water solution; they neutralize acids to form salts and have pH values more than 7.0. Common examples are sodium hydroxide (NaOH: caustic soda) and ammonium hydroxide (NH_4OH: ammonia water). Portland cement concrete made with nonalkali-reactive aggregates is highly resistant to strong solutions of most bases. It is unaffected by continuous exposure to 10 percent solutions of sodium or potassium hydroxides. However, if circumstances are such that sodium hydroxide can penetrate concrete and become concentrated at an evaporating face, physical damage may result from crystallization of $Na_2CO_3 \cdot 7H_2O$ or $Na_2CO_3 \cdot 10H_2O$ formed by reaction between the NaOH and carbon dioxide from the air.[31] Alternate wetting and drying of concrete by NaOH solution may bring about the same effect.

Calcium, ammonium, barium, and strontium hydroxides are harmless. Penetration by $NaCO_3$ solutions, with subsequent concentration at an evaporating face, or alternate wetting and drying, may result in crystallization of sodium carbonates without involving CO_2 from the air. These effects are minimized by use of concrete of high quality, that is, low permeability. Solutions of sodium carbonate have no deleterious effect on well-cured good concrete (except as mentioned above) but the surface of fresh concrete may be deteriorated.

7.8.4.3 Salts. The chlorides and nitrates of ammonium, magnesium, aluminum, and iron all attack concrete with those of ammonium being most harmful. However, ammonium carbonate, fluoride, and oxalate are not harmful, as these form relatively insoluble compounds upon reaction with hardened cement paste.

Alkali Soils and Waters. Soluble sulfates, principally those of sodium, calcium, and magnesium, occur naturally in many parts of the U.S. Soils or waters containing these sulfates are often called "alkali" soils or waters. All are potentially harmful to concrete. The attack is usually accompanied by an expansion due to the formation of one or more solid reaction products having a volume greater than that of the solids entering into the reaction. Thus sodium sulfate can react with the hydrated calcium aluminate in hardened portland to produce ettringite (See Section 8.1) which has a larger volume. Sodium sulfate can also react with calcium hydroxide to produce gypsum which has a volume more than double that of the initial solids. Solutions of 0.5 percent of sodium, potassium, or magnesium sulfates may strongly attack concrete with the attack of magnesium sulfate being more extensive than that of the other two.

Local conditions will greatly influence the potential for sulfate attack on concrete. A soil which is relatively high in sulphates, but which is dry for long periods of the year, or which is so drained that the solutions formed by the leaching of the soil are removed from the vicinity of the concrete, will be much less destructive than one in which the sulphate content is lower, but the physical conditions are more favorable. Regions in which alternate wetting and drying occur fairly frequently are likely to be particularly destructive. It is also often found that the salt content of waters and soils varies widely between points quite close together; this increases the difficulty of assessing in advance the conditions to which a concrete will be subjected. The salt content of the soil is only important insofar as it represents the reserve supplies available for maintaining, or increasing the salt content of the waters contained in it, or draining from it. Despite these difficulties, an examination in suspected cases of the general ground conditions affords a guide to the precautions which are required to ensure the permanency of a concrete.

The U.S. Bureau of Reclamation[31] has classified sulphate soils in relation to the degree of sulphate attack as shown in Table 7.8. Lea[2] also published some guidance (Table 7.9) for concrete which is used in or around sulphate soils.

Ice Removal Agents. The use of chemical agents, usually calcium chloride or common salt, on exposed concrete pavements to keep them free of ice has given rise to a serious problem of surface scaling commonly called "salt" scaling. The exact mechanism through which de-icers cause scaling has not been determined. Powers[72] claimed that the salts produced osmotic pressure which causes movement of water towards the top layer of the slab where freezing and subsequent damage can take place. Verbeck and Klieger,[73] in observing that the greatest scaling damage occurred when the concrete was exposed to relatively low concentrations of salts (2 to 4 percent solution), believed the attack to be primarily physical and not chemical in nature.

The more severe damage occurs when concrete is subjected to alternating freezing and thawing with the de-icer solution remaining on top of the pavement rather than being replaced with fresh water prior to each re-freezing.[73] If the liquid is removed from the surface of the concrete prior to re-freezing, no scaling takes place. Concrete pavements and bridge decks newly placed are particularly subject to de-icer scaling, even when properly air entrained, and it is therefore of practical importance to provide adequate curing periods. In general, the application of ice removal agents should be delayed until the concrete has been in place for at least 6 weeks at a temperature above 40 F.[61] Other factors that will help prevent scaling from de-icer chemicals are:[71]

a. Using cement contents not less than 620 lb/cu yd.
b. Limit slump to not more than 3 in.
c. Use good, sound, clean aggregates.

d. All proportioning, mixing, placing, and finishing are to be done in accordance with approved standards.

e. Using adequate amounts of entrained air (varying from 5 percent for large aggregate to 7-1/2 percent for small aggregate).

Non-air-entrained concrete, regardless of temperature, curing time, or type of cement, does not usually exhibit satisfactory resistance to scaling.

7.8.4.4 Water. The waters draining from mountain regions are often almost free from salts but are acidic owing to the presence of carbon dioxide (carbonic acid) and humic acid arising from decaying organic matter. The amount of humic acid that can be carried by water is small and though it has some adverse effect on concrete, it is much less serious than that of very pure or carbon-dioxide containing waters.

Pure water is capable of leaching out lime from hardened cement compounds. When carbon dioxide is present in the water in a free state, it increases the aggressive action of the water. Waters which contain only combined carbon dioxide present as carbonates or bicarbonates have no appreciable solvent action. It is only when the water is either very pure or contains free carbon dioxide that it attacks hardened concrete.

Waters which are acid owing to the presence of organic acids and carbon dioxide do not normally produce more than a surface attack on portland cement concrete, although with prolonged exposure, the depth of attack may penetrate several inches. Free sulfuric acid can also occur in water, and if the water is continuously renewed, the attack may be considerable, especially on lean or permeable concrete. The attack by sulfuric acid is partly due to its acidic nature and partly due to deleterios reactions involving the sulfate ion.

Soft waters may also have a solvent action on concrete, but the process is rather complicated and is dependent on a number of interrelated factors. Lea[2] has suggested that waters with a temporary hardness above 4-5 parts $CaCO_3$/100,000 are not likely to be very aggressive unless the free carbon dioxide content is very high, that is, about 5 parts/100,000 or above. As the temporary hardness decreases, so does the concentration of carbon dioxide required for the water to be aggressive. For a temporary hardness of 1-2 parts/100,000, a water will not have a marked solvent action unless the free CO_2 is above about 1 part/100,000; for a temporary hardness of 0.5 part the corresponding CO_2 value is about 0.5 part, while waters of very low temporary hardness, below say 0.25 part/100,000 can be aggressive even if the free CO_2 content is negligible. In general, the pH value is not a safe guide but waters may be aggressive at pH values up to 7, or even 7.5, if the temporary hardness is below about 0.25 part/100,000 and at pH values up to 6.0 to 6.5 for higher values of temporary hardness.

Concrete pavements within a few miles of the ocean can be subjected to attack by seawater. The usual modes of deterioration are by corrosion of reinforcement and attack by salts. The chemical action of seawater on concrete is mainly due to the presence of magnesium sulphate. As noted in Section 7.8.4.3 (Alkali Soils and Waters) a solution of 0.5 percent of magnesium sulfate will strongly attack concrete by forming disrupting, swelling reaction products. General expansion of concrete in seawater is usually not observed, however, and it is believed that the presence of chlorides in the seawater retards the swelling of the concrete. Lea[2] views the chemical action of seawater as one of several reactions proceeding concurrently. Leaching actions remove lime and calcium sulphate while reaction with magnesium sulphate leads to the formation of ettringite which may cause expansion, rendering the concrete more open for further attack and leaching. The relative contributions to deterioration of expansion and leaching will depend on the conditions at hand.

The rate of chemical attack by seawater is increased by temperature and both the rate and its effects are influenced by the type of cement. With high-quality concrete and, particularly for warm or tropical waters, an appropriate choice of cement, the deterioration from chemical action can be reduced to a very slow rate. If the cement used contains not more than 8 percent C_3A and if the concrete is of high quality and adequately consolidated, it should have excellent resistance to seawater. The aggregates used should be nonexpansive with alkalies, but if this requirement cannot be met, the cement should contain not more than 0.60 percent alkalies (as Na_2O).

7.8.4.5 Oils. Oils can be classified as either mineral oils or vegetable and animal oils. In general, mineral oils do not affect hardened concrete while vegetable and animal oils do.

Mineral Oils. Mineral oils, such as gasoline, fuel oils, and petroleum distillates in general do not attack mature hardened concrete but may affect the hardening of freshly mixed concrete. Creosotes which contain phenols, cresols, and similar acidic compounds may, however, have some effect on concretes. Lubricating oils which are entirely of mineral origin do not attack concrete, but if vegetable oils are present, they have a definite adverse action on the concrete. Lea[2] contends that it is impossible to make a concrete impermeable to lighter mineral oils (viscosities below 0.06 poises at 20 C) such as diesel oil, kerosene, and petrol.

Vegetable and Animal Oils and Fats. Although the concrete contained in pavements will probably not be exposed to any great degree to vegetable and animal oils and fats, the reader should be aware that these materials can also adversely affect concrete.

Vegetable and animal oils and fats are natural products composed mostly of the glycerides or other esters of the higher members of the several series of fatty acids, but they also contain, in some cases, notable amounts of the corresponding free fatty acids and alcohols. Glycerine (or glycerol) is the chief alcoholic constituent of oils and fats. Oils of vegetable origin usually contain appreciable quantities of free fatty acids. Animal fats, when freshly rendered, contain only small quantities of free acid, but the amount is increased on exposure to the atmosphere.

The glycerides and other esters are broken up by hydrolysis into their constituent alcohol and acid components. This process, termed *saponification*, can be produced by the action of acid or alkali solutions. When oils are brought into contact with concrete the free lime present in the hardened cement saponifies the material, forming a calcium salt of the fatty acid and liberating polyhydric alcohol. This alcohol can itself often react with lime. Using glyceryl oleate (olein) in olive oil as an example, calcium oleate and glycerol form, and the glycerol then combines further with more lime to form calcium glycerolate. This is typical of the destructive mechanism of saponifiable oils and fats on concrete. If free acids are also present, as is often the case, they also attack the concrete and form their calcium salts.

Glycerine is completely miscible with water in all proportions and is a solvent for lime. It attacks concrete by slowly combining with, and dissolving, any free calcium hydroxide present in hardened cement. A solution of glycerine in water as weak as 2 percent has a destructive action on fresh portland cement concrete, but has little effect on a well-carbonated surface of a mature concrete. Stronger solutions of 10 percent and upwards have a very destructive effect on hardened concrete.

The extent to which an oil can attack a concrete is dependent on the ease with which it can penetrate the material. The viscosity of the oil is thus an important factor and, other things being equal, any action is likely to be the less serious, the more viscous the oil. Exposed oils which have accumulated moisture and undergone oxidation become more active in attack.

Among the more common nonmineral oils are cottonseed oil, palm oil, rapeseed oil, olive oil, coconut oil, linseed oil, tung oil, lard oil, and fish oils. It should be assumed that, in general, all such oils are likely to be harmful to unprotected portland cement concrete, though in some cases it is possible to render their attack negligible. Some drying oils such as linseed oil and tung oil, when not supplied in a continuous or intermittent manner to the concrete, tend to undergo oxidation and harden when exposed to air, thereby eliminating the probable rancidity due to decomposition which would in turn develop certain oxidized free fatty acids. Without this rancidity, they should not harm the concrete and in fact are used for surface treatments of concrete (See Chapter 11).

7.9 ABRASION RESISTANCE

Abrasion resistance of concrete is described as the ability of a concrete surface to resist being worn away by rubbing and friction.[1] This wearing action will definitely occur from vehicular traffic on pavements.

It has been generally found that the resistance of concrete to abrasion improves with increasing compressive strength of the concrete.[74-77] In a study of concrete ranging in strength from 2500 to 10,000 psi, it was found that the abrasion resistance increased somewhat linearly with compressive strength with the higher strength concrete having a five-fold increase over the lower strength concrete.[74]

Another important factor in determining the abrasion resistance of concrete is the hardness of aggregate used in the concrete.[74,78] The service life of concrete subjected to abrasion can be greatly lengthened if hard and tough aggregates are used, especially in the coarse aggregate fraction. In the range of concrete strengths from 3000 to 5000 psi, the effect of the hardness of coarse aggregate is significant but above 8000 psi, it is minimal.[71] The effect of a very hard aggregate, calcined bauxite, is discussed in Chapter 12.

Other factors affecting surface hardness are curing, construction practices, and surface treatments. A well-cured concrete can produce a surface up to twice as wear resistant as a surface cured for only 3 days.[79] A minimum of 14 days moist curing has been recommended.[61] The construction practice of using unvented salamanders or other unvented heaters during cold weather concreting operations may produce carbon dioxide gas which will be highly detrimental to the development of surface hardness.[61]

Surface treatments in the form of liquids are described in Chapter 11. Other liquids such as magnesium or zinc flurosilicate or sodium silicate can also be used to improve surface hardness.[61] The addition of "dry shakes," in the form of dust or small aggregate pieces similar to those described in Section 12.6, to fresh concrete can also provide improved surface resistance. Liquid surface treatments are more effective on moderate or low-strength concretes, concretes which have been over troweled, troweled or finished too early, or which have not been cured. Floating and troweling the concrete before it has lost its surface water sheen or all free water on the surface will adversely affect the development of surface hardness. Dust coats (dry shakes) have been highly effective in improving surface hardness and their use should conform to manufacturers' recommendations.

7.10 IMPACT RESISTANCE AND DYNAMIC CONDITIONS

The impact resistance of concrete has received attention from investigators for many years. Abrams,[80] in 1917, ran dynamic tests on concrete which indicated that the strength of concrete increased as the rate of applying load increased. Jones and Richart,[81] in 1938, found similar results. Other work, reported that same year by Glanville,[82] concluded from the relation between the rate of loading and the shape of the stress-strain curve for concrete, that the amount of plastic strain decreased as the rate of loading increased.

Watstein,[83] in 1953, using 3- by 6-in. plain concrete cylinders subjected to impact loading found that with increasing loading rates there was an increase in strength, modulus of elasticity, strain energy absorbed, and total strain. Lundeen,[84,85] in 1963 and 1964, found similar results for the same size specimens.

Extensive tests on impact strength of concrete were also made in 1964 by Green.[86] In general, Green found that the higher the static compressive strength of the concrete, the lower the energy absorbed per impact blow (pendulum hammer loading) before cracking. On the other hand, however, he found that the impact strength increased with compressive strength (and therefore age) at a progressively increasing rate. His work also indicated that this relation is different for each coarse aggregate and storage condition of the concrete. For the same compressive strength, the impact strength is greater for coarse aggregate of greater angularity and surface roughness. Concrete made with gravel coarse aggregate has a low impact strength with failure taking place due to insufficient bond between mortar and coarse aggregate. When the aggregate is rough, the concrete is able to develop the full strength of much of the aggregate in the region of failure. The use of fine sand usually leads to a slightly lower impact strength.

Cowell,[87] in 1966, concluded from the results of impact tests on concrete that the mechanical properties of concrete increased as the rate of loading increased. He noted that the energy absorption also increased as the rate of loading increased. Atchley and Furr,[88] in 1967, found similar results but noted that there was evidence that the amount of increase becomes a constant value at the higher rates of loading. They also observed increases in the secant modulus of elasticity and strain as the rate of loading increased. There was evidence in their work that a concrete specimen behaved visco-elastically under dynamic loading.

In general, increases of 30 to 80 percent in impact strength over static compressive strength of concrete have been reported. This increase in strength partially explains the greater ability of concrete to absorb strain energy.

7.11 FATIGUE STRENGTH

In practice the failure of concrete takes place over a range of stress rather than as an instantaneous phenomenon. This is particularly evident when repeated loading is applied such as is the case with traffic over pavements and bridges. When concrete fails under a number of repeated loads, each smaller than the static compressive strength, failure in fatigue is said to have taken place.

The fatigue strength of concrete is lower than the static strength of concrete. This behavior is generally attributed to the formation and progressive development of cracks.[89-92] This cracking is extensive and the observed strain at failure is much larger than in static failure.[33] The nonelastic strain increases with the number of cycles and has been reported to be as high as 4000×10^{-6} after 13 million cycles at 200 cycles per minute. Cyclic loading also affects lateral strains, with the Poisson's Ratio progressively decreasing with increasing number of cycles.[89]

The fatigue strength of many engineering materials is described by a fatigue limit, that is, a strength where the material will be able to withstand an infinite number of load cycles. Concrete does not generally have a fatigue limit but instead has fatigue strength specified for a large number of cycles, such as 10 million or more. A fatigue strength for 10 million cycles of flexural-tension loading such as is common to pavement loading is generally assumed to be 55 percent of the static strength of the concrete.[33] Slightly higher fatigue strengths have been reported for cyclic compression loading. Most investigators believe the failure mechanism is the same in both tension and compression. The number of repetitions that concrete can resist decreases rapidly as the maximum stress increases to higher and higher percentages of the static strength of the concrete. Murdock[91] noted that fatigue failure occurs at the same fraction of ultimate strength, and is thus independent of the magnitude of this strength and the age of the concrete. He attributed the failure to a single parameter, that is, the deterioration of bond between the cement paste and aggregate with repeated loadings. For a stress equal to 70 percent of the static strength, approximately 5000 repetitions have been reported as causing failure.[93] Murdock[91] also suggested that fatigue failure occurs at a constant strain, independent of the applied stress level or the number of cycles necessary to produce failure.

Air-entrained concrete and lightweight concrete have the same fatigue behavior as concrete made with ordinary aggregate.[94]

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CHAPTER 7

PORTLAND CEMENT CONCRETE

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TABLE 7.1

Compound Composition of Portland Cements[31]

Type of Cement	Compound Composition, Percentage							
	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO	MgO	Ignition Loss
Type I	49	25	12	8	2.9	0.8	2.4	1.2
Type II	46	29	6	12	2.8	0.6	3.0	1.0
Type III	56	15	12	8	3.9	1.3	2.6	1.9
Type IV	30	46	5	13	2.9	0.3	2.7	1.0
Type V	43	36	4	12	2.7	0.4	1.6	1.0

TABLE 7.2

Comparison of the Rates of Gaining Strength of Concrete Cured at Various Temperatures[38]

Age, Days	Compressive Strength, psi				
	5F	40F	70F	100F	200F
1	---	---	1005	---	---
3	1555	1740	2530	3280	3730
5	1545	2335	3410	4260	4470
7	1395	2730	3940	4650	4670
14	1415	3600	4520	5440	3650
28	1460	4140	5440	6200	3370
49	---	---	---	---	3430

TABLE 7.3

Effect of Size on Compression Specimen on Indicated Strength of Concrete[31]*

Size of Cylinder, in.	Relative Compressive Strength, %	Size of Cylinder, in.	Relative Compressive Strength, %
2 x 4	108	12 x 24	91
3 x 6	106	18 x 36	86
6 x 12	100	24 x 48	84
8 x 16	97	36 x 72	82

*Adopted from Figure 227, Reference 31.

TABLE 7.4

Effect of Height-Diameter Ratio of Compression Specimen on the Indicated Strength of Concrete[44]

Ratio of height to diameter of cylinder or core	Strength Correction Factor
2.00	1.00
1.75	0.99
1.50	0.97
1.25	0.94
1.00	0.91

TABLE 7.5

Influence of Aggregate Content on the Coefficient of Thermal Expansion[55]

Cement-aggregate Ratio	Linear coefficient of thermal expansion at the age of 2 years, 10^{-6} per degree F
Neat Cement	10.3
1:1	7.5
1:3	6.2
1:6	5.6

TABLE 7.6

Coefficient of Thermal Expansion of 1:6 Concretes Made With Different Aggregates[56]

	Linear coefficient of thermal expansion 10^{-6} per deg F		
Type of Aggregate	Air-cured Concrete	Water-cured Concrete	Air-cured and wetted Concrete
Gravel	7.3	6.8	6.5
Granite	5.3	4.8	4.3
Quartzite	7.1	6.8	6.5
Dolerite	5.3	4.7	4.4
Sandstone	6.5	5.6	4.8
Limestone	4.1	3.4	3.3
Portland Stone	4.1	3.4	3.6
Blast Furnance Slag	5.9	5.1	4.9
Foamed Slag	6.7	5.1	4.7

TABLE 7.7

Reduction in Permeability of Cement Paste with the Progress of Hydration[60]

Age, Days	Permeability cm/sec
Fresh	2×10^{-4}
5	4×10^{-8}
6	1×10^{-8}
8	4×10^{-9}
13	5×10^{-10}
24	1×10^{-10}
Ultimate	6×10^{-11} (calculated)

Note: Water-cement ratio = 0.7

TABLE 7.8

Attack on Concrete by Soils and Waters Containing Various Sulfate Concentrations[31]

Relative degree of sulfate attack	Percent water-soluble sulfate (as SO_4) in soil samples	ppm sulfate (as SO_4) in water samples
Negligible	0.00 to 0.10	0 to 150
Positive[1]	0.10 to 0.20	150 to 1,500
Severe[2]	0.20 to 2.00	1,500 to 10,000
Very severe[3]	2.00 or more	10,000 or more

1. Use type II cement.

2. Use type V cement, or approved portland-pozzolan cement providing comparable sulfate resistance when used in concrete.

3. Use type V cement plus approved pozzolan which has been determined by tests to improve sulfate resistance when used in concrete with type V cement.

TABLE 7.9

Concrete for Sulphate Soils[2]

Total SO_3 in soil[3] per cent	SO_3 in ground-water parts per 100,000	Cement Type	Concrete Needed: Minimum cement content lb/yd^3	Concrete Needed: Maximum free w/c ratio*
0-2	30-120	Ordinary Portland or Portland blast-furnace,	550	0·50
		Sulphate-resisting Portland,	475	0·55
		Supersulphated	515	0.50
0·5-1·0	120-250	Sulphate-resisting Portland, Supersulphated or High Alumina	550	0·50
1-2	250-500	Sulphate-resisting Portland or Supersulphated,	625	0·45
		High Alumina	575	0·45
Over 2	Over 500	(i) High alumina (ii) Sulphate-resisting Portland or Supersulphated with surface protection	625	0·40

*Total water in concrete less that absorbed by the aggregate.

CHAPTER 8

EXPANSIVE CEMENT CONCRETE

INTRODUCTION

8.1 BACKGROUND

The origin of expansive cements is usually associated with the discovery of ettringite by Candlot[51] in 1890. This compound is described chemically as $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$ and is formed during the hydration of expansive cements. It is generally believed to be the source of the expansive force in the cement and is comparable to the natural mineral of the same name. In older literature, it is often called the "cement bacillus." In 1892, Michaelis[254] suggested that ettringite was responsible for the destructive expansion of portland cement concretes in the presence of sulfate and sea waters. Attempts were made in the 1920's in France to control the expansive energy of ettringite formation but were unsuccessrul.[147] Lossier[210] began experimentation with expansive cements in 1936 with the objective of chemically prestressing concrete. He is generally credited with the discovery of expansive cements. Klein,[166] at the University of California, Berkeley, pursued the development of an anhydrous calcium sulfoaluminate as a way to control ettringite-formation reactions and patented it as an invention on 18 October 1961. Another patent was granted to Klein,[162] in 1966 as a continuation-in-part of the first. Expansive cements made under the Klein patents are the principal expansive cements in use in the United States today. A more detailed development of expansive cements can be found in reports by Aroni, et al.,[15] Kalousek,[147], Li,[189] Mather,[228] North,[291] and ACI Committee 223 report.[2]

Concrete, when placed in an environment where it begins to dry out and lose moisture, will begin to shrink. The amount of drying shrinkage which occurs in concrete depends on the characteristics of the cement, aggregates and admixtures, mix proportions, mixing times, placement methods, time between adding water and placing, temperature and humidity, member size and configuration, slump, and curing. When a pavement or structural member is restrained during drying shrinkage by the subgrade friction, reinforcement or other portions of the structure, tensile stresses develop. Often the drying shrinkage is sufficiently large to induce stresses which exceed the tensile strength of the concrete causing cracks. The frequency and size of the cracks depend on the amount of shrinkage and restraint.

Expansive cements, however, when properly used in concrete that receives adequate curing, will cause that concrete to increase in volume after hardening. When properly restrained by reinforcement or other means, compressive stresses are induced in the concrete. If the expansive potential of the concrete is small, only small compressive stresses are developed. Subsequent drying shrinkage will relieve these compressive stresses instead of creating tension, hence cracking will be minimized. If the expansive potential of the concrete is great, the initial compressive stresses will be large and after subsequent drying shrinkage, a significant amount of stress will still remain, thus effectively producing chemically prestressed concrete.

This section of the report is concerned with the characteristics and applications of expansive cement concretes as they pertain to pavements. The bibliography on expansive cements and expansive cement concretes contained in this chapter was developed from numerous sources and many of the items cited were not actually reviewed. It is the most comprehensive bibliography developed to date on this subject, however, and is presented for use by individuals desiring to examine this subject in more detail.

8.2 DESCRIPTION OF MATERIALS

8.2.1 Expansive Cements. Expansive cements are cements which, when mixed with water, form a paste that, after hardening, tends to increase in volume to a significantly greater degree than portland cement paste.[4] They can be used to compensate for volume decrease due to shrinkage or to induce tensile stress in reinforcement (post-tensioning). These types of expanisve cement have been produced in the United States and have been referred to as Types K, M, or S.[2] Currently (1976) only Type K cement is being manufactured in the United States.

Type K expansive cement is a mixture of portland cement, anhydrous tetracalcium trialuminate sulfate ($C_4A_3\overline{S}$),* calcium sulfate ($CaSO_4$), and lime (CaO). The $C_4A_3\overline{S}$ is a constituent of a separately burned clinker that is interground with portland cement or alternately, it may be formed simultaneously with the portland cement clinker compounds during the burning process.[4]

Type M expansive cement consists of interground or blended mixtures of portland cement, calcium aluminate cement, and calcium suitably proportioned. The Type M expansive cement that was produced in the United States is not the same as the stressing cement (SC), often called Type M, produced in the Soviet Union, also from portland cement, calcium aluminate cement and gypsum. The SC product is proportioned in such a manner that quick setting, fast hardening, and high early strength are obtained and, therefore, it is generally not used in conventional concrete.[4]

Type S expansive cement is a type of portland cement containing a large computed tricalcium aluminate (C_3A) content and interground with an amount of calcium sulfate above the usual amount found in portland cement.[4]

All expansive cements consist predominately of portland cements (about 70 to 94 percent). The chemical compositions are similar to those of portland cement except for higher sulfate contents (4.5 to 6.5 percent S), generally higher alumina contents (5 to 9 percent A), and generally lower silica contents.[147] The lower the amount of the expansive agent the greater is the similarity in compositions. Table 8.1 shows some typical chemical analyses of these cements.

8.2.2 Expansive Compounds. Several products of a calcium sulfoaluminate (CSA) composition have been produced and marketed in Japan for the purpose of adding them to concrete to cause expansions. These

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* where $C = CaO$, $A = Al_2O_3$ and $\overline{S} = SO_3$, $S = SiO_2$

expansive components are similar to the component used in Type K cement and cause the concrete to expand through formation of ettringite. Table 8.2 shows a typical chemical analysis of two Japanese expansive components.[170] Table 8.3 shows the analyses of the mixtures of the expansive components and Type I portland cement.[170] The rationale behind the use of the components is "... to make possible importation of desired expansion quantities to concretes at the required times by varying the dosages of components according to the quality of cement, mix proportion of concrete, dimensions of concrete members, ambient temperature, exposure conditions of concrete, and other conditions."[170] A similar approach is used in the United States with a United States-produced self-stressing cement in tunnel construction.[131]

8.2.3 Expansive Cement Concrete. The concrete made with any of the expansive cements and the concrete made with additions of the CSA will be called expansive cement concrete for the purposes of this report and should not be confused with such materials as "nonshrink" concrete which contain materials which produce a gas during the unhardened state of the concrete, thus causing it to swell before hardening. The expansion in expansive cement concrete takes place after hardening.

Concretes made with expansive cements have been classified as shrinkage-compensating concrete and self-stressing concrete.[2] Shrinkage compensating concrete is an expansive cement concrete which when properly restrained by reinforcement or other means will expand an amount equal to or slightly greater than the anticipated drying shrinkage of the concrete. Because of the restraint, compressive stresses will be induced in the concrete during expansion. Subsequent drying shrinkage will reduce these stresses but ideally, a residual compression will remain in the concrete thereby eliminating shrinkage cracking.[2,4] Self-stressing concrete is an expansive cement concrete in which the expansion, if restrained, will induce a compressive stress of high enough magnitude to result in a significant residual compression in the concrete after drying shrinkage has occurred.[2] Concretes made with the expansive component, CSA, can be classified as either shrinkage-compensating or self-stressing depending on the amount of CSA used and the performance of the concrete after hardening. Figure 8.1 shows the simplified expansion behavior of both shrinkage-compensating and self-stressing cements and indicates the effect of the amount of expansive component of high enough magnitude to result in a significant residual compression in the concrete after drying shrinkage has occurred.[289] Concretes made with the expansive component, CSA, can be classified as either shrinkage-compensating or self-stressing depending on the amount of compressive stress developed in the concrete, which in turn is related to the amount of expansive component (and hence expansion) and restraint of the concrete. Figure 8.2 shows the calculated relation between expansion, percent reinforcement, and concrete compressive prestress. Figure 8.3 shows the effect of the amount of expansive component on expansion with time. Kalousek[147] and many others have reported that the compressive stresses induced by shrinkage-compensating cements in concrete are from 25 to 100 psi, while self-stressing cements may produce stresses from 100 to 1000 psi.

MECHANICAL PROPERTIES

In general, the physical properties of expansive cement concretes are related to the amount and rate of expansion and the amount and direction of restraint. Other factors such as type of curing and mixture ingredients also influence this behavior. For the shrinkage-compensating cement presently produced in the United States, the physical properties of shrinkage-compensating concretes are comparable to those of portland cement concretes of similar mixture proportions.

Some degree of prestress can usually be achieved through the use of expansive cements. The magnitude of that prestress, as shown in Figure 8.2, is related to the amount of expansion. For United States-produced expansive cements, the cement producer controls the expansive potential of the cement by adjusting the chemical composition and fineness of the cement. When the expansive component is used, the supplier or user can control the expansive potential by varying the amount of component added. Once the expansive potential of the cement is determined, however, other factors will then influence the rate and amount of expansion the expansive cement achieves. These factors include cement content, type of aggregate and admixture used, mixing time, type of curing, temperature during mixing or curing or both, and degree of restraint provided by steel reinforcement or other means.

The data discussed in the following sections tends to relate mostly to Type K cements but this is not intentional. Type K cement has had much broader use and promotion that Types M and S, and substantially more information on its behavior is available. Where no statement is made regarding a specific property for Types M or S or for the CSA component, it is due to a lack of data.

8.3 FACTORS AFFECTING EXPANSION

8.3.1 Cement Content. The selection of the cement content is usually governed by the strength requirements of the project and is determined by tests of the expansive cement concrete made with the actual materials to be employed on the job. The procedures used for selecting the mixture proportions of the expansive cement concrete are similar to those used with normal portland cement concrete. In addition to the strength requirement, there will also be an expansion requirement. When using expansive cements with a built-in expansive potential, the higher the cement content of the mixture, the larger the expansion for a given degree of restraint. This can be seen in Figure 8.4 for zero percent reinforcement. The use of the expansive component, CSA, allows the cement content to remain constant while the expansions increase due to higher concentrations of the CSA in the mixture. Figure 8.3 shows this effect for a cement content of 670 lb per cu yd when the CSA concentration is increased from 11 to 17 percent.

8.3.2 Aggregate Type. The normal concrete aggregates used for portland cement concretes can also be used in expansive cement concretes. This is true for lightweight and heavyweight aggregates, also. The type of aggregate used will, however, affect the shrinkage characteristics of the concrete (as it also does in normal concrete) and the expansive behavior.[81] Figure 8.5 shows the effect of structural lightweight aggregate concrete expansions compared to an equally proportioned and sized normal weight concrete.[12,14] Figures 8.6 and 8.7 show the effects of three aggregate types used with a Type K self-stressing cement on the longitudinal expansion of some triaxially restrained concrete. The observed differences have been attributed to the availability of water contained within the lightweight aggregates which caused the early-age expansion to be higher than for two normal weight aggregates. Unpublished WES data on mortars made with both concrete sand and saturated lightweight sand confirm this finding. In Figure 8.6 the concrete with the lightweight aggregate continued to expand during the early stages thus resulting in significantly lower drying shrinkage and subsequent loss of compressive prestress due to the drying shrinkage. Lower drying shrinkage was also observed (Figure 8.7) when the specimens were moist cured to full expansion for 33 days. Data of Epps and Polivka[81] (Table 8.4) showed that after a 1-year drying period, shrinkage-compensating concrete containing river gravel had a residual compression (0.028 percent of expansion) whereas the use of sandstone resulted in a slight tension (0.019 percent of shrinkage).

Aggregates containing gypsum or other sulfates may increase expansions and cause delayed expansion and/or later disruption of the concrete. Significant amounts of chlorides in aggregates, such as found in beach sands, tend to decrease expansion and increase drying shrinkage. For these reasons, it is recommended that job aggregates be used in laboratory trial mixture tests to see if this problem exists.[4]

It has also been suggested[2] that aggregate size may affect expansions but that this variable may be clouded by the proportioning changes associated with changing aggregate sizes.

8.3.3 Admixtures. Most of the admixtures commonly used in portland cement concrete have been used in expansive cement concrete. The effects of air-entraining agents, water-reducing agents, retarding agents, and accelerator admixtures on the expansion of a specific type or brand of shrinkage-compensating cement may be either beneficial or detrimental.

Before use on the job, all admixtures should be tested in trial mixtures with job materials and proportions under simulated ambient conditions. In general, however,[4]

a. Air-entraining admixtures are as effective with shrinkage-compensating concretes as with portland cement concretes in improving freeze-thaw and deicer salt durability.

b. Water-reducing admixtures may be incompatible with shrinkage-compensating concretes due to acceleration of the ettringite reaction which usually has the effect of decreasing expansion.

c. Calcium chloride when used as an accelerator usually reduces expansion and increases shrinkage.

d. Fly ash and other pozzolans may affect expansions and also influence strength development. They should be used with caution, particularly at low temperatures.

e. Set-retarding water-reducing admixtures may react similar to water-reducing admixtures (see b above).

Since the methods of mixing and placing can influence admixture performance, laboratory results alone may not always correlate with job results.

8.3.4 Mixing Time. The formation of ettringite begins as soon as water is added to the cement during mixing. The amount of ettringite that can be formed is predetermined by the proportioning of the constituents of the cement. Because actual expansions of the concrete will not begin until the concrete has reached a final set, it is wholly possible, through extended mixing and agitation, to cause the rate of ettringite formation to be accelerated to the point when most of the ettringite will form in the unhardened mixture and thus greatly reduce the expansive potential of the mixture. Figure 8.8 shows the effect of mixing time on the unrestrained expansion of a Type K shrinkage-compensating concrete. Polivka[309] recommended that during hot weather construction, the concrete be mixed and placed within 45 minutes. A longer time can be tolerated at normal or lower temperatures.

8.3.5 Curing. In expansive cement concrete, curing is of even more importance since it has a great influence on its expansion characteristics. Water is needed to aid in both the formation of the strength-giving calcium silicate hydrates and the formation of the expansion producing ettringite. Larger expansions will be obtained when the concrete is water cured rather than sealed with curing compounds or covered by a moisture-proof cover such as polyethylene sheeting (Figure 8.9). Although membrane curing is considered adequate, water curing is preferred.

8.3.6 Temperature. Temperature affects the rate of ettringite formation and hence will affect both the rate and magnitude of expansion. Concretes mixed at higher temperatures will have their ultimate expansions reduced because of the high rate of ettringite formed during mixing and the rapid increase in strength gain of the concrete which then contributes to restraining the expansion. Concretes mixed at lower temperatures will have their ultimate expansion reduced because of the lower rate of ettringite formation and higher creep deformation at early ages. This behavior is shown in Figure 8.10.[309]

8.3.7 Restraint. Restraint of expansion can be applied by either external or internal means. The degree of restraint has a significant influence on the measured expansion. Unrestrained expansions may be as great as an order of magnitude over restrained expansions. To fully utilize the steel reinforcement, the concrete should be permitted to expand in order to develop tension in the steel and hence compression in the concrete. Restraint by subgrade friction or adjacent structures also develops compressive stress in the concrete but very little, if any, tensile stress in the steel.[309] Such restraint may help to reduce the cracking tendency of the concrete but it is not as efficient or reliable in minimizing cracking as in the case where the concrete is permitted to expand. A typical relationship between the amount of restraint (percent steel) restrained expansion of the concrete and compressive stress developed is shown in Figure 8.11 for a Type K self-stressing concrete.[309]

8.4 COMPRESSIVE, FLEXURAL, AND TENSILE STRENGTH

8.4.1 Shrinkage-Compensating Concrete. Shrinkage-compensating concretes develop compressive, tensile, and flexural strengths equivalent in rate and magnitude to Type I or II portland cement concretes. Unpublished WES data from samples from all cement mills in the United States producing Type K shrinkage-compensating cement indicate that compressive strengths at 28-days age of Type K shrinkage-compensating cement concrete are from 5 to 10 percent greater than Type II portland cement concrete made with exactly the same proportions. No information was avilable as to whether this behavior was the same for flexural and tensile strengths.

8.4.2 Self-Stressing Cements. It has been generally established that the compressive strength of self-stressing concretes is inversely related to the cement of expansion and that the amount of expansion is inversely related to the amount of restraint.[2] Figure 8.12 shows the effect of degree of restraint on expansion of concrete made with Type K cement.[29] Figure 8.13 shows the effect of restraint on compressive strength of Type K self-stressing cement concrete.[29] High strength self-stressing concrete can readily be made when proper consideration is given to the many variables associated with the use of this cement.

8.4.3 Expansive Components. The strength of portland cements modified with the addition of CSA will depend, among other things, on both the amount of component and the degree of restraint as mentioned in Section 8.4.2. Figure 8.14 shows the relationship between expansion and compressive strength ratio in unrestrained expansion specimens for varying amounts of two CSA products.[170] Of course, once reinforcing steel is introduced into the concrete, some chemical prestress can then occur. Figure 8.15* shows the relationship between induced chemical prestress and compressive strength for concretes of varying CSA content. Figure 8.16* shows the

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*Data from Denki Kagaku Kogyo Kabushiki Kaisha, Tokyo, entitled "Basic Properties of Mortar and Concrete Made with DENKA CSA," Jan 1973, and "Mechanical Characteristics of Chemically Prestressed and Steam-Cured Elements Made with DENKA CSA," Sep 1972.

amount of increase in flexural stress at the time of cracking with increasing chemical prestress in CSA modified concretes. The strength behavior of CSA modified portland cement concretes can be expected to be similar to that of the United States produced Type K cement concretes of either the shrinkage-compensating or self-stressing type depending on the amount of CSA used.

8.5 DIMENSIONAL CHANGES

The expansion behavior of expansive cement concretes has already been discussed in the previous sections. The drying shrinkage of these concretes is not a function of expansion and depends on the usual parameters such as richness of mixture, water/cement ratio, aggregate type used, etc. In general, the drying shrinkage characteristics of a shrinkage-compensating concrete are similar to those of a Type I portland cement concrete.

The loss of prestress force due to shrinkage and creep must be taken into consideration as in mechanically prestressed applications. The magnitude of stress losses in steel and concrete due to drying shrinkage and creep have been observed to be approximately equal to or less than those observed for conventional prestressed concrete.[25] Klein and Bertero[163] reported on a "precreep" mechanism at work in Types K and M self-stressing concrete. This mechanism reduces the ultimate creep strain to values considerably less than those of conventional concrete subjected to the same ratio of sustained stress to ultimate stress at the same ages of loading. They postulated that this resulted from the application of load to the concrete at early age through the self-stressing mechanism when its ultimate strength is low.

Data available on the creep characteristics of expansive cement concretes indicate that their creep coefficients are within the same range as those of portland cement concretes of comparable quality. The coefficient of thermal expansion of expansive cement concretes is consistent with that of corresponding portland cement concretes made with the same materials.

8.6 DURABILITY

When properly designed and adequately cured, expansive cement concretes made with Type K, S, or M are comparably resistant to freezing and thawing and deicer scaling as portland cement concretes. The effect of air and aggregates are essentially the same.

The type and particularly the composition of the portland cement portion of a shrinkage-compensating, self-stressing, or CSA modified cement can have a significant effect on the durability of the concrete to sulfate exposure. Expansive cements made with a Type I or III portland cement may be under-sulfated with respect to the aluminate available and therefore susceptible to further expansion and possible disruption after hardening when exposed to an external source of additional sulfates. This problem becomes even more acute with CSA modified cements when the CSA additions are large. On the other hand, the expansive cements made with Type II or Type V portland cement clinker and adequately sulfated, produce concretes having sulfate resistance equal to or greater than portland cement made of the same type.

8.7 ABRASION RESISTANCE

The abrasion resistance of expansive cement concrete is not significantly different than that of a comparable portland cement concrete made with similar proportions and ingredients.

8.8 TOUGHNESS

The toughness of unreinforced concrete should not be significantly changed by the use of expansive cements in liu of normal portland cement. In reinforced elements with levels of expansion such that the concrete develops a prestress force, the toughness will be improved over similarly reinforced portland cement concrete elements as additional loading and deformations will be necessary to overcome this prestress force.

8.9 FATIGUE STRENGTH AND DYNAMIC CONDITIONS

The dynamic response of expansive cement concrete should not be significantly different from comparable portland cement concretes. It has been speculated that the fatigue behavior of expansive cement concretes should be improved over that of portland cement concrete on the basis that initial no-load cracking of the material will be less and a small compressive prestress may exist even in unreinforced concrete. No data have been found to substantiate this, however.

8.10 PERMEABILITY

The restrained expansion of expansive cement concretes produces a more dense matrix which serves to reduce the permeability and absorption. The absence or reduction of cracking will greatly reduce the ingress of fluids into the concrete.

USE OF EXPANSIVE CEMENT CONCRETE

Several uses of expansive cement concrete in pavements have been reported.* The author is aware of pavements which have been built in California, Connecticut, Minnesota, Ohio, and Wisconsin. Information on most of them involves construction, and performance data are scarce.

8.11 UNREINFORCED PAVEMENT

Two experimental sections of concrete pavement near Palmdale and Lodi, California, were installed in 1963 using shrinkage-compensating concrete without steel reinforcement.[20, 188,350] The expansive cement used was of the shrinkage-compensating type with only 15 percent expansive component. The only restraint provided was by the subgrade friction and end anchors. This experiment was not successful. The cracking in the expansive cement concrete was not less than that of the control sections made of normal portland cement concrete. Aroni, et al.[15] concluded that for successful shrinkage compensation of highway pavements, a larger percentage of expansive component and some reinforcement would be required. It is now recommended that, as a minimum, cement contents of

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* References 20, 68, 80, 111, 115, 116, 188, 199, 226, 283, 284, 316, 324, 338, 339, 342, 350, 351, 352, 359, 361, and 369.

515 lb of shrinkage-compensating cement per cubic yard of concrete and 0.15 percent reinforcement be used.[4]

8.12 WIRE FABRIC REINFORCED PAVEMENT

A one-fourth-mile section of pavement made with Type K shrinkage-compensating cement was placed outside of Columbus, Ohio, in 1964.[20,116, 342] Contraction joints were begun at the normal spacing of 60 ft and then gradually increased to 68, 70, 77, 90, 94, 119, and 120 ft. The contraction joints were sawed and dowel basket construction incorporated. Usual construction techniques were used including membrane curing. The cement content was 611 lb cement per cubic yard of concrete. The pavement used 6 x 4-00/4 wire fabric reinforcement. In the winter when temperatures fell, the sawed joints opened and fine hairline cracking developed directly below the sawed joint. The joints were placed in the center line of the dowel baskets, and due to the lack of restraint from reinforcing at these points, continuous restraint was lost in the slab at that point. Crack widths were substantially less than would be expected in conventional potland cement. After four years of service, no visible intermeadiate cracking had occurred.

From 1969 to 1972 several sections of taxiway at Love Field, Dallas, Texas, were replaced using Type K shrinkage-compensating concrete.[306] The first section complete was 5200 ft long, 75 ft wide, and 14 in. thick. Reinforcement was welded wire fabric with longitudinal steel #000 gauge on 6-in. centers and transverse sheet #3 gauge on 12-in. centers. The steel was placed four inches down from the top of the slab. Dowels were 1-1/2-in. diameter smooth bars 24 in. long. Transverse joints were spaced at 125 ft. If Type I cement had been used, the joints would have been spaced 50 ft apart. The remaining sections of taxiways constructed were similar in dimensions to the first, except the thickness was increased to 16 in. and the transverse joint spacing reduced to 75 ft. The joint spacing was reduced because of observed hairline cracks which occurred midway between contraction joints on the section having a 125-ft joint spacing. The general response of many individuals who have viewed these sections over a 4- to 6-year period are that they are performing much better than would have been expected if they had been made with Type I or II portland cement.

8.13 CONTINUOUSLY REINFORCED PAVEMENT

8.13.1 Shrinkage Compensating Concrete. Another expansive cement concrete pavement was placed in Ohio in August 1965.[20,116,342] This was a 1350-ft length of continously reinforced pavement without joints near Mansfield, Ohio. This pavement contained the same amount of Type K cement (611 lb per cu yd) as the Columhus pavement but used 0.68 percent reinforcement. A Type I portland cement concrete pavement was placed the same day for comparisons sake. After the first 200 ft of shrinkage-compensating concrete had been placed, work was temporarily halted due to rain. Inspection of the pavement one year later revealed no visible cracks in the first 200-ft section, but the remaining 1150 ft had 54 cracks compared to 181 cracks in a 1167-ft section of Type I portland cement concrete. The widths of the shrinkage-compensating concrete cracks were less than one-half the width of those which occurred in the portland cement section. A further inspection one year later (two years after placement) showed that the cracks had increased to 9-ft intervals in the expansive cement section and to 5-ft intervals in the control pavement. The 9-ft interval cracks had also become wider than the 5-ft interval cracks. It was speculated that the compressive prestress force in the steel had diminished thus allowing the cracks to widen or end movement of the total section had occurred and the pavement behaved much like a Type I portland cement concrete or both.[2] Since 200 ft had been placed without cracks, it was concluded that the length of pavement was a governing factor due to thermal contraction and subgrade friction.

8.13.2 Self-Stressing Concrete. A prototype pavement slab 185 ft long, 13 ft 4 in. wide, and 6 in. thick without joints was constructed at the prestressed concrete plant of C. W. Blakeslee and Sons, Inc., in Hampden, Connecticut in 1963.[15,189,359] A self-stressing cement was used at a cement content of 940 lb per cu yd. The longitudinal reinforcement was twelve 1/2-in. diameter, and six 3/8-in. diameter, 7-wire, high-tensile strands with end anchorage. This represented a longitudinal steel percentage of 0.23 percent. The transverse steel was 1/2-in. diameter deformed bars on 8-in. centers for 0.53 percent steel. Water curing was applied for seven days. Overall expansion at 3 days was 0.27 percent. No change was observed between 5 and 14 days. This expansion corresponded to a chemical prestress in the concrete of 150 psi. Concrete and steel expansions were in close agreement indicating no apparent slippage. Although never subjected to traffic, the slab was used as a casting bed at the plant. At an age of 23 months, it was reported to be crack-free with a good surface appearance and no wear or deterioration had been observed.[20]

The Blakeslee slab served as a pilot for three experimental sections constructed on Connecticut State Route No. 2 near Glastonburg in 1963.[339] Each section was approximately 500 ft long, 24 ft wide, and 6 in. thick. The concrete was placed on a base of 2-in. screenings and 1-in. bituminous concrete. Between the bituminous concrete and the expanding concrete there were two layers of polyethylene sheeting to minimize subgrade friction. The cement was Type K and was used at a cement content of 800 lb per cubic yard. The amount of expansive component in the cement was 35 percent. The longitudinal reinforcement consisted of twenty-one 1/2-in. diameter, 7-wire, high tensile strands with end anchorages for a longitudinal steel percentage of 0.175 percent. The transverse steel consisted of 7/8-in. diameter bars on 24-in. centers and hooked at each end for a steel percentage of 0.42. Each cable was initially tensioned with an approximate 1000-lb draw between 18 WF 55 steel beams imbedded at 500-ft intervals in a 20-ft by 24-ft by 9-in. sleeper slab. Severe contruction difficulties were encountered due to a very rapid loss of workability of the concrete. The difficulties were magnified by a lack of adequate consolidation equipment during placement. The first slab placed experienced a possible compressive failure at approximately the midpoint of the slab. This failure became evident shortly after finishing. It was theorized that the retarders which had been added to the mixture to resolve the workability loss problem had kept the strength gaining characteristics of the expansive cement lagging behind the stress build-up due to expansion and that failure occurred during early stages. Strain measurements were made on the steel, and dimension changes in both directions were noted. The average longitudinal and transverse expansions of the concrete were 0.10 and 0.05 percent, respectively. This translated to concrete stresses of 25 to 100 psi longitudinally and 140 psi transversely. The low longitudinal expansion was attributed to subgrade friction and to larger early age expansion in concrete made with more water. Inspection at one year found a reasonably large number of cracks. Most of them ran across the entire width of the pavement and were prevalent in the center area of each slab. In the middle third of the slab, cracks were found as close as 10 to 20 ft apart. At the slab ends few cracks were noticed. The cracking pattern and measured expansions indicated that the prestress was less effective in the center section of the slabs than at the ends. It was assumed that the cracking resulted from an inadequate development of full prestressing which in turn resulted from lack of ultimate expansions.[2] Although this project experienced many problems, much was learned from it. Most significant was that the optimum length for chemical prestressing probably lies between 185 ft (Blakeslee slab) and 500 ft.

Nagataki and Yoneyama[284] reported on the construction of a 300-ft-long 13-ft, 4-in. wide and 8-in.-thick continously reinforced pavement at Niigata, Japan, in 1968 which used a CSA expansive component in portland cement. A similar control slab was also constructed. Each pavement used 548 lb of cement per cubic yard with the expansive cement concrete pavement also using a 15 percent addition of CSA. The longitudinal reinforcement percentage was 0.68. Curing was provided for 14 days. Both slabs were instrumented with imbedded strain measuring devices. The response from these instruments over a period of 600 days indicated that the unit length changes and displacement of the expansive concrete pavement were completely different from that of normal concrete. It was apparent that the expansion of concrete due to the use of expansive admixture compensated for the contraction of the pavement due to temperature drop and to drying shrinkage. Calculating the compressive stresses in in the pavement from the observed strains. Nagataki and Yoneyama[284] found that "self-stresses" varied from 370 psi in sections of no-displacement where both subgrade and reinforced contribute to restraint to maximum values of 200 psi permanent "self-stress" where only the reinforcement contributed to restraint and time losses had occurred. After four years, no cracks had been found in the expansive concrete pavement while the conventional concrete pavement was observed to have cracking after 56-days age.

8.14 FIELD PERFORMANCE SUMMARY

The initial applications of expansive cement to pavements have met with varied success. As with any new material or concept, this is to be expected until more experience and expertise is developed. A large number of structural slabs and slabs-on-grade have been constructed using the shrinkage-compensating cement, and most have been very successful at eliminating all drying shrinkage cracks. Since sawed joints, construction joints, and shrinkage cracks are all detrimental to long-term performance, no fixed limits have been placed on size of placements. Slabs with areas greater than 20,000 sq ft have been successfully placed, but it is recommended[4] that placements of 7000 to 12,000 sq ft be used out-of-doors where temperature changes are large.

Slabs with a 1 to 1 ratio of length to width have been ovserved to give the best performance,[2] but ratios of 2 to 1 or 3 to 1 are more common and have been successfully used. Contraction joint spacings of 100 ft in exposed areas are recommended.[4]

The need for internal restraint by reinforcing steel was highlighted by the unsuccessful unreinforced California highways. The use of self-stressing cement in continuously reinforced pavements appears to have excellent potential as is evidenced by the fact that joint spacings of 300 ft have been obtained.[284]

8.15 EASE OF CONSTRUCTION

The unhardened characteristics of expansive cement concrete are sufficiently similar to concretes made with Types I or II portland cement so that no special techniques or equipment are required for satisfactory placement. Successful placements have been made by wheelbarrow, ready-mix truck, bucket, conveyor, paving train, and shotcrete.

The same placing recommendations as for portland cement concretes are equally important for expanisve cement concretes. ACI Committee 223[4] recommends that some additional precautinos also be followed to ensure adequate expansions and satisfactory results. These include measures to prevent subgrade and forms from taking moisture from the freshly placed concrete, proper positioning of reinforcement, avoiding of delays during placing, careful use of vapor barriers, and keeping concrete temperatures and mixing times within acceptable limits. These precautions were suggested for shrinkage-compensating concrete and are even more important when self-stressing concrete is being used.

In general, expansive cement concretes have more cohesiveness than portland cement concretes and less tendency to segregate. This cohesiveness or "fat" provides excellent finishing qualitites. Its behavior is similar to air-entrained concrete including the same stickiness, but this usually presents no problems. Similarly, there is little or no bleeding even though a relatively high slump may be used. Due to lack of bleed water, however, there is a tendency for finishers to start too soon. On the other hand, in warm weather, shrinkage-compensating concrete will typically set faster than Type I or Type II portland cement concretes, and finishing may start somewhat sooner than normal. For these reasons, finishing may require greater manpower for a shorter period of time than would be typical for the usual concrete finishing operations under similar conditions.

Adequate curing is essential for all expansive cement concretes. Shrinkage-compensating concrete requires continuous curing at moderate temperatures for several days after final finishing operations to prevent early drying shrinkage and to develop strength, durability, and other desired properties. Any deficiencies in the method of curing may also reduce the amount of initial expansion which is needed to offset later drying shrinkage. The usually accepted methods of curing are satisfactory for shrinkage-compensating concrete; however, those that provide additional moisture to the concrete such as ponding, continuous sprinking, and wet coverings are preferred to ensure adequate water for ettringite formation and expansion. For self-stressing cements, those systems which provide additional moisture are essential to ensure that the desired expansions occur. Curing of shrinkage-compensating concrete should be continued for a minimum of seven days. The minimum period of curing for self-stressing concrete may have to be a few days longer because of the greater quantities of ettringite which will form. Unpublished data at WES indicate that the expansions of self-stressed mortar are approximately 95 percent complete in 9 to 10 days.

Curing of concrete pavements should commence immediately after final finishing. It may be necessary to fog spray or cover the surface of the concrete temporarily if other methods of curing are delayed, especially in hot, dry, or windy weather. If a liquid curing membrane is used, it is recommended[4] that it be applied in two directions, at a coverage rate suggested by the manufacturer, immediately following the final finishing as it progresses. To accomplish this, power spray equipment capable of covering large areas more rapidly should be used rather than small, portable spray tanks. Expansive cement concretes should also be protected during the initial curing period against extremes of temperatures during either cold or hot weather periods. Again, this is more critical for self-stressing concretes than for shrinkage-compensating concretes.

8.16 EASE OF MAINTENANCE

The use of expansive cements in pavements should result in a relatively more maintenance free pavement than conventional concrete pavements. Cracking should be significantly reduced in both amount and crack width. The pavements, whether made with shrinkage-compensating concrete or self-stressing concrete, will probably not be crack free, however. Joint spacings will be significantly increased thus resulting in fewer joints and a fewer number of problems associated with joint maintenance.

Damage to pavements made with expansive cement concretes will be more detrimental than similar damage to conventional pavements due to the loss of compressive forces or chemical prestress in the pavement. As these pavements will probably have been constructed with greater joint spacings or thinner sections, the loss of these forces will precipitate further damage in the pavement under formerly acceptable levels of load or temperature variations. More elaborate repair systems and techniques than presently used on conventional pavements will be needed but are all within the scope of present knowledge on repair.

8.17 ENVIRONMENTAL COMPATIBILITY

The environmental compatibility of expansive cement concrete pavements should not be any different than that of conventional concrete or prestressed concrete pavements.

8.18 AVAILABILITY AND COST

As noted in Section 8.2.1, Type K expansive cement is the only expansive cement in production in the United States, although Types S and M have been produced and marketed as recently as a few years ago. At the writing of this report (summer 1976), there are 17 United States cement mills producing Type K shrinkage-comepnsating cement and one mill producing self-stressing cement. The Japanese produced CSA expansive component for cement is also available through Japanese outlets in the United States and can be used to make either shrinkage-compensating or self-stressing concrete.

There should be no additional equipment costs involved in the use of expansive cement concrete for pavements as existing equipment for use with conventional concrete is very suitable for its batching, mixing, placing, finishing, curing, and texturing. The basic cost of the cement may be slightly greater than for Type I or II portland cement, depending on the volumes involved. The magnitude of difference diminishes with increasing volumes. Any additional cost in cementing materials will probably be compensated for in savings in construction costs or in reduced sections. Increased joint spacings mean less jointing materials and joint preparation. Chemically prestressed pavements may result in thinner cross sections for the same levels of anticipated loading. The economic justification for using expansive cement concrete must be developed by costing the entire concreting operation and not just the materials cost.

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CHAPTER 8

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Table 8.1

Typical Chemical Analyses for United States Produced Shrinkage-Compensating Cements*

Chemical Analysis %	Type K		Type M	Type S	
	A	B		A	B
SiO_2	18.4	19.0	19.1	18.2	17.2
Al_2O_3	6.6	5.8	8.5	7.4	9.1
Fe_2O_3	2.0	1.9	2.1	2.0	1.8
CaO	60.0	61.5	61.1	62.9	63.9
MgO	3.1	4.1	1.9	0.8	1.2
SO_3	6.3	6.3	4.4	5.4	5.6
Ignition loss	2.8	1.5	2.2	1.9	0.8
Total					

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* Unpublished data from WES participation in ASTM Cooperative Test Program on Shrinkage-Compensated Cement, 1970.

Table 8.2

Test Results of CSA Expansive Components[170]

Chemical Analysis, %	Component	
	A	B
SiO_2	4.0	1.3
Al_2O_3	10.0	7.2
Fe_2O_3	1.2	0.4
CaO	52.5	59.7
MgO	0.6	0.6
SO_3	28.3	30.2
Ignition loss	1.0	0.6
Total	99.0	100.0
Specific surface, cm^2/g	2280	2250
Specific gravity	2.93	3.04

Table 8.3

Chemical Compositions of Mixtures of Expansive Components and Type I Portland Cement[170]

Chemical Analysis %	Type I Portland Cement	8% Cement Replacement with Component A	7% Cement Replacement with Component B
SiO_2	22.8	20.8	20.8
Al_2O_3	4.4	4.9	4.6
Fe_2O_3	3.1	3.0	3.0
CaO	64.5	63.5	64.2
MgO	1.6	1.5	1.5
SO_3	1.9	4.0	3.9
Ignition loss	0.5	0.5	0.5
Total	98.9	98.9	99.1

Table 8.4

Effect of Aggregate Type on Length Change of Shrinkage-Compensating Concrete[309]

Aggregate Type	14-Day Expansion, percent	1-Year Contraction, percent	Residual Length Change, percent
River Gravel	0.080	0.052	+0.028
Sandstone Aggregate	0.065	0.084	-0.019

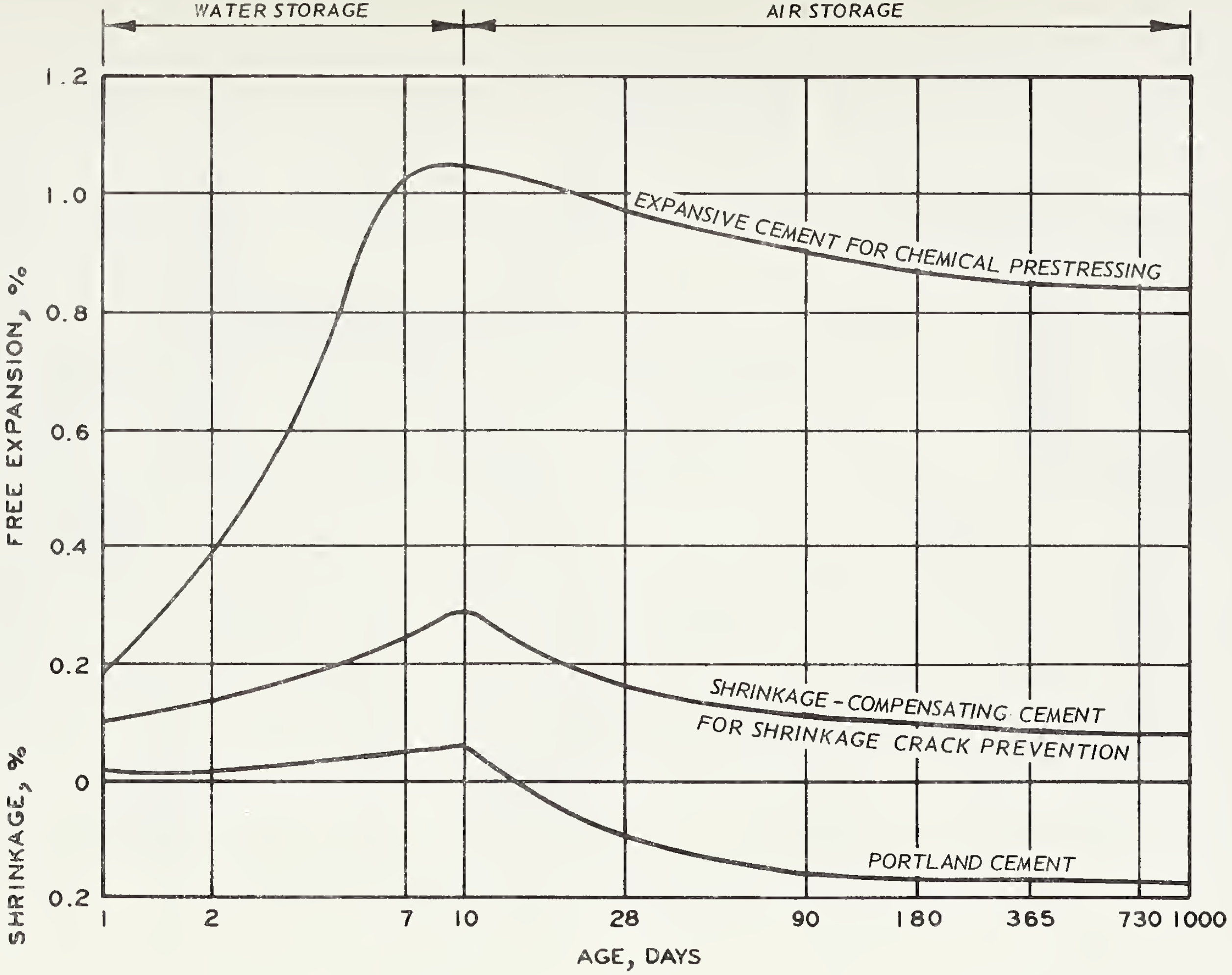

Figure 8.1. Volume changes of neat cement pastes[289]

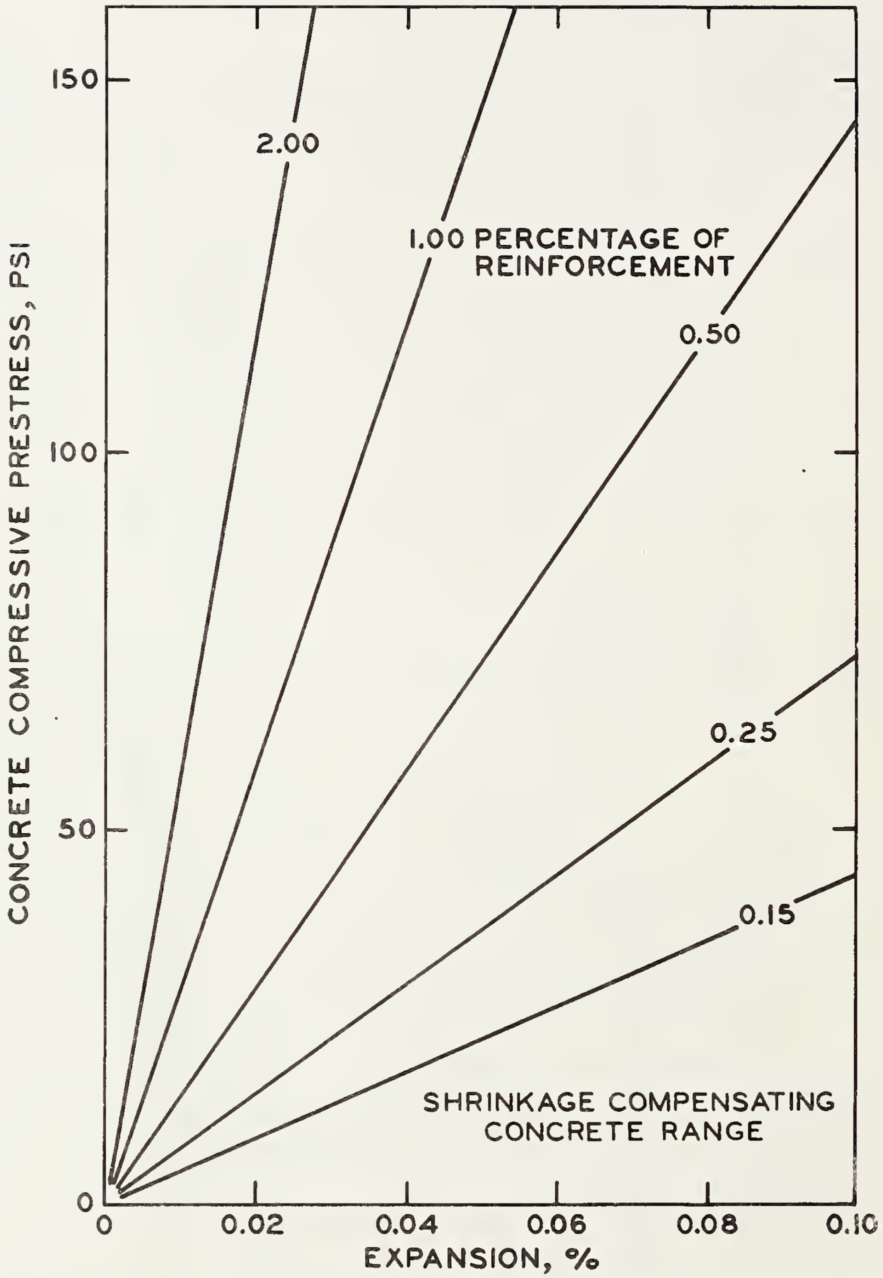

Figure 8.2. Calculated compressive stresses induced by expansion[4]

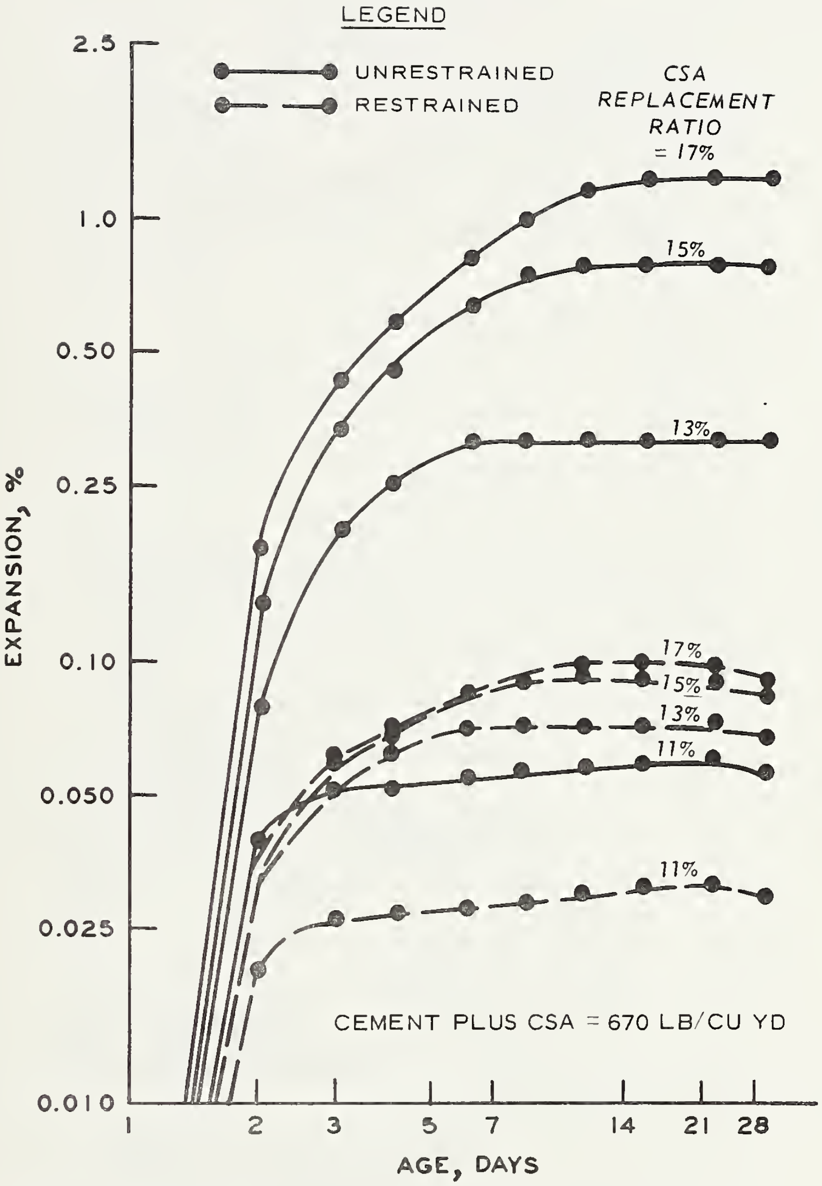

Figure 8.3. Influence of restraint on expansion of concrete using CSA component[170]

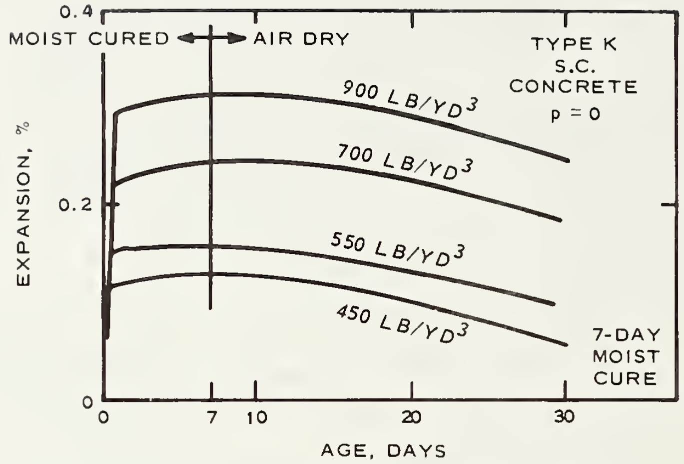

Figure 8.4. Effect of cement content on the expansion characteristics of type K cement shrinkage-compensating concrete[2]

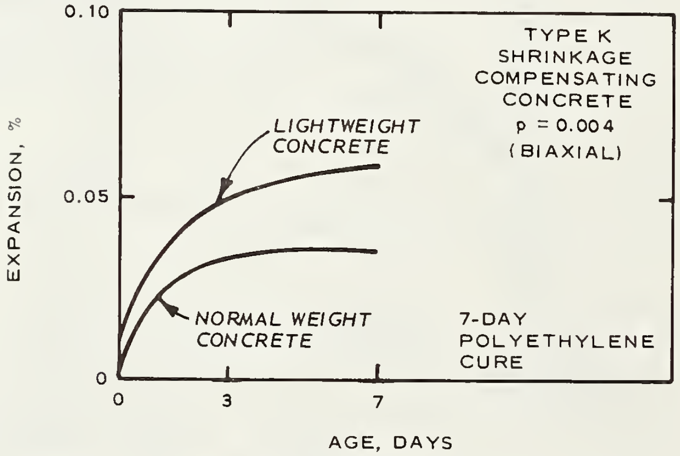

Figure 8.5. Effect of lightweight aggregate versus normal weight aggregate on the expansion of type K cement shrinkage-compensating concrete[12,14]

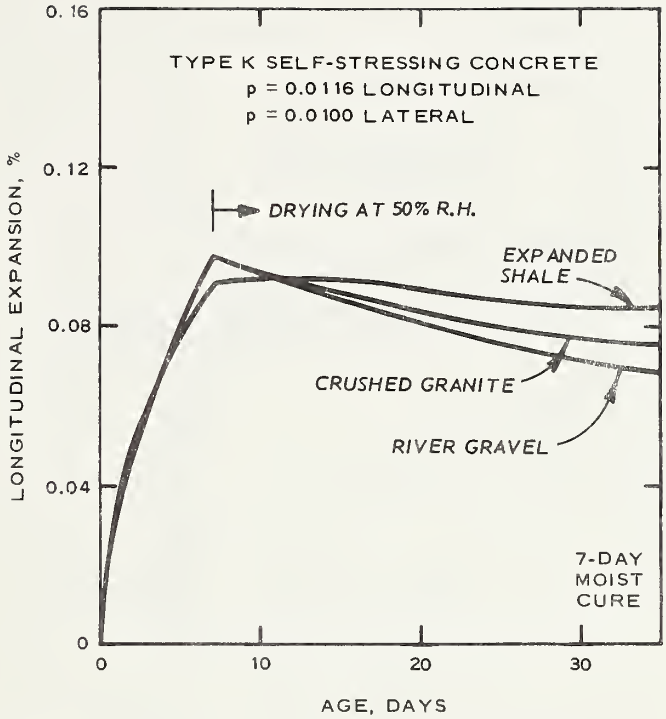

Figure 8.6. Effect of aggregate type on longitudinal expansion of triaxially restrained concretes moist cure 7 days[14]

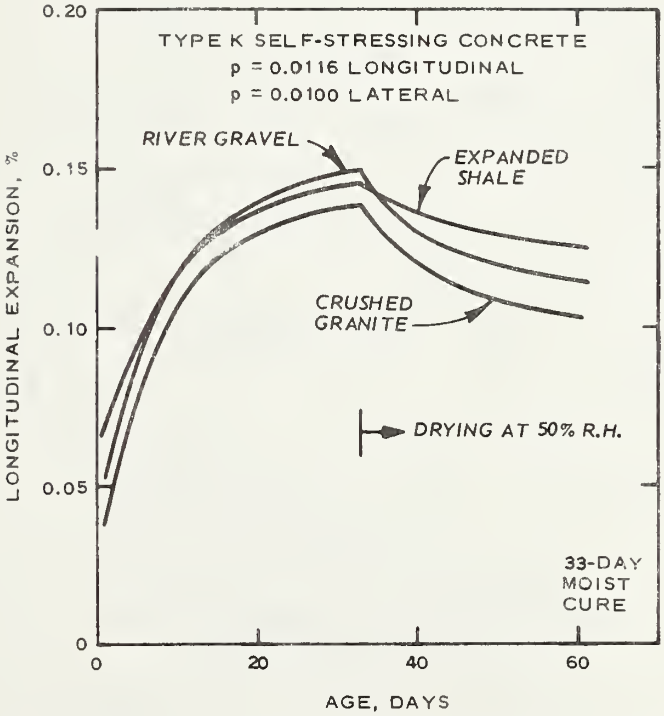

Figure 8.7. Effect of aggregate type on longitudinal expansion of triaxially restrained concretes moist cured 33 days[14]

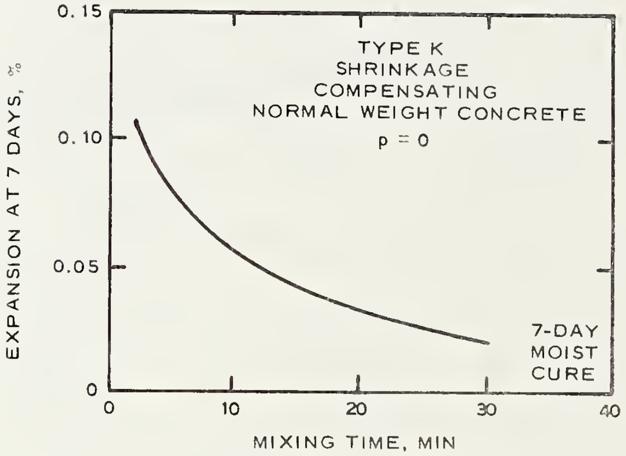

Figure 8.8. Effect of mixing time in the laboratory on expansion of unrestrained type K[2] cement shrinkage compensating concrete

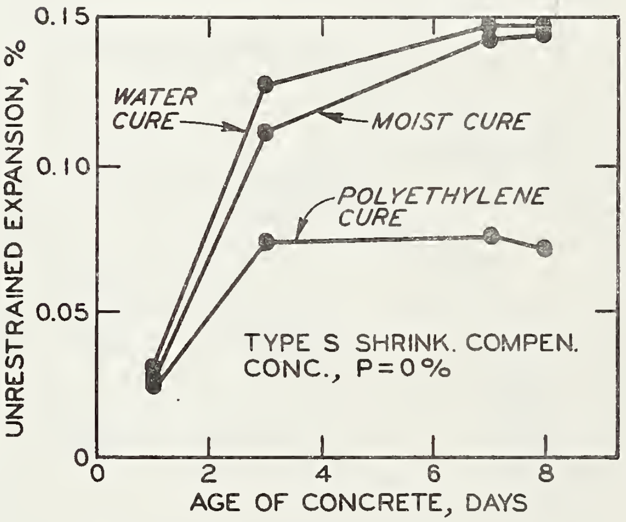

Figure 8.9. Effect of curing on expansion of unrestrained type S cement shrinkage compensating concrete[309]

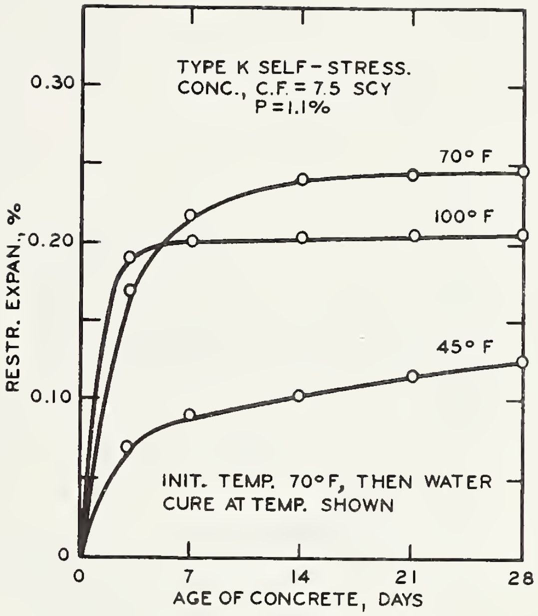

Figure 8.10. Effect of curing temperature on rate and amount of expansion of type K self-stressing concrete[309]

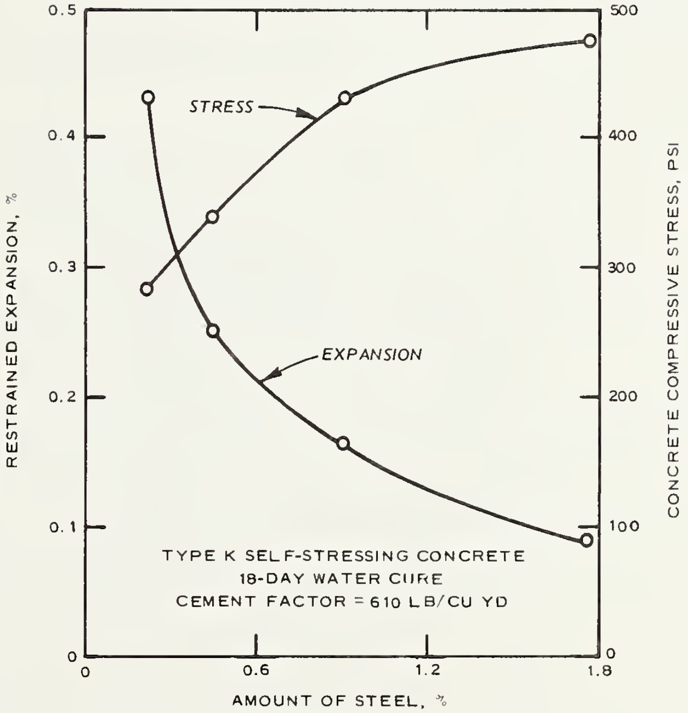

Figure 8.11. Effect of restraint on expansion and compressive stress of type K self-stressing concrete[309]

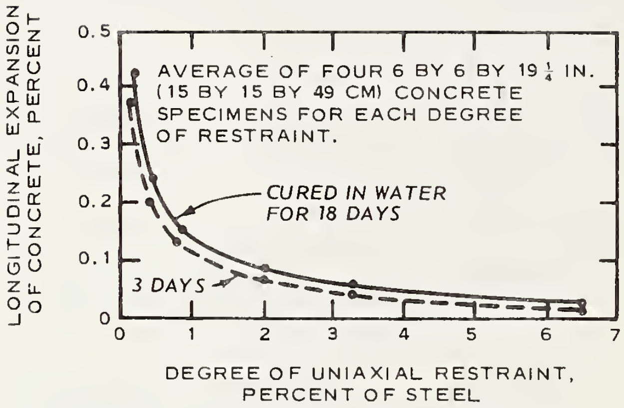

Figure 8.12. Effect of degree of restraint on expansion of concrete made with type K cement at 18 days[29]

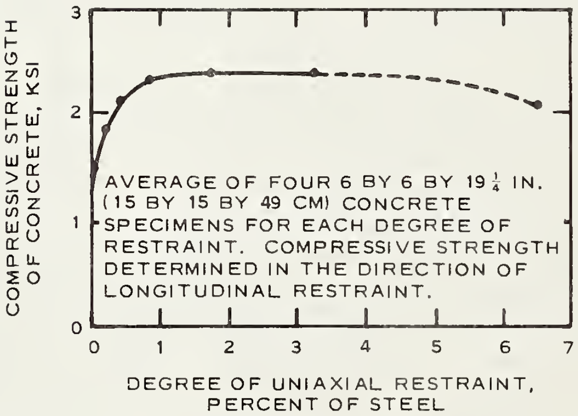

Figure 8.13. Effect of degree of restraint on compressive strength of self-stressing concrete made with type K cement at 18 days[29]

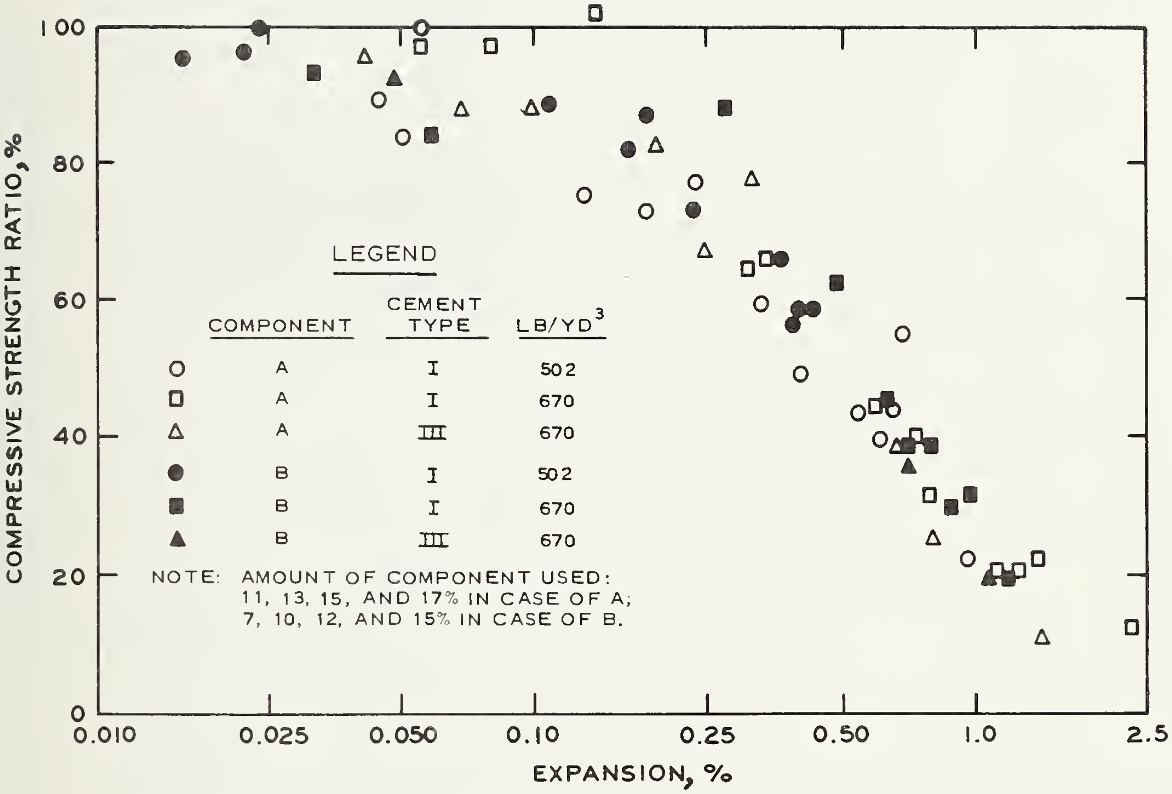

Figure 8.14. Relations between expansion and compressive strength ratio in unrestrained expansion specimens[170]

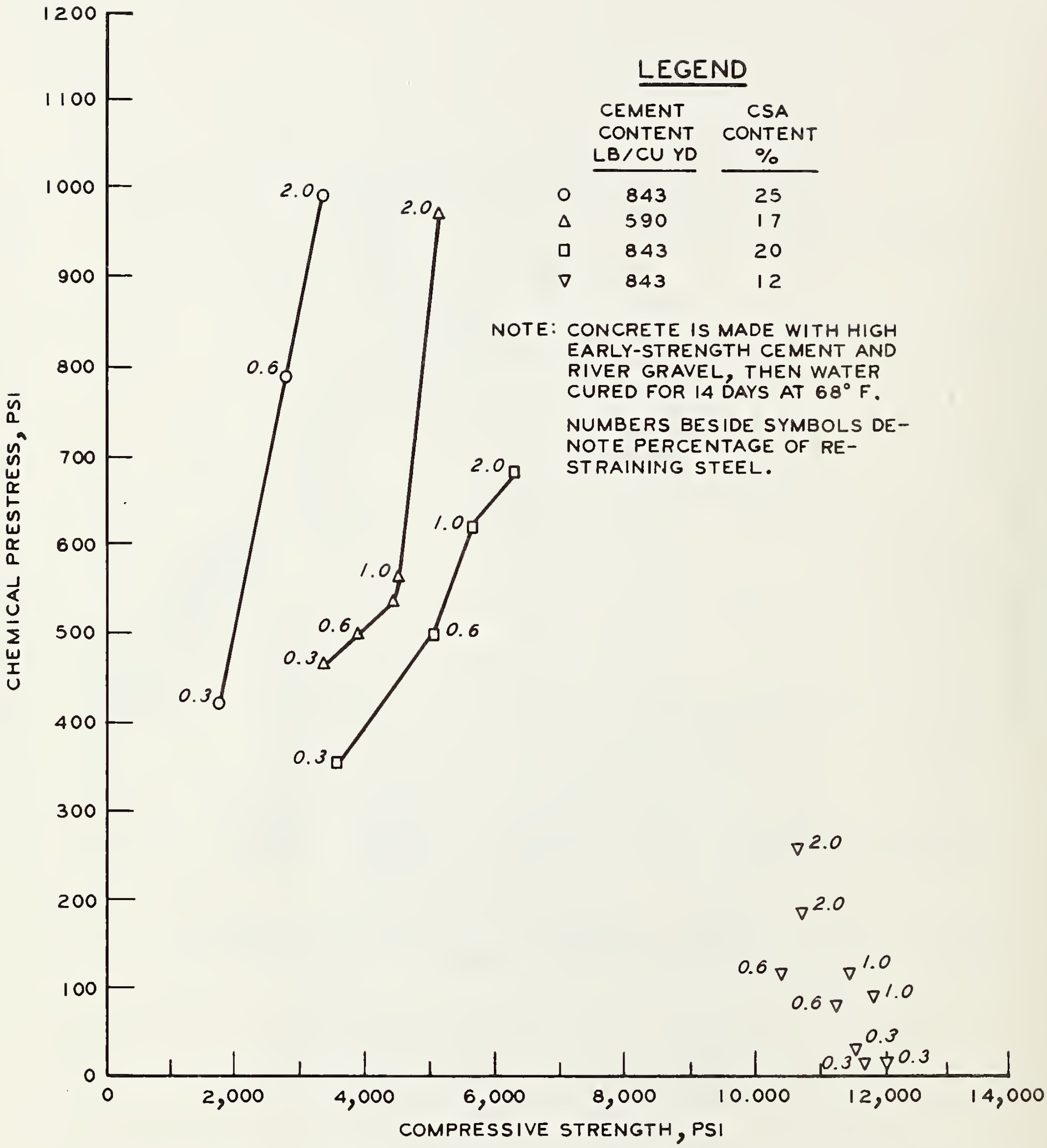

Figure 8.15. Relation between induced chemical prestress and compressive strength for CSA concrete*

*See page 210 for reference.

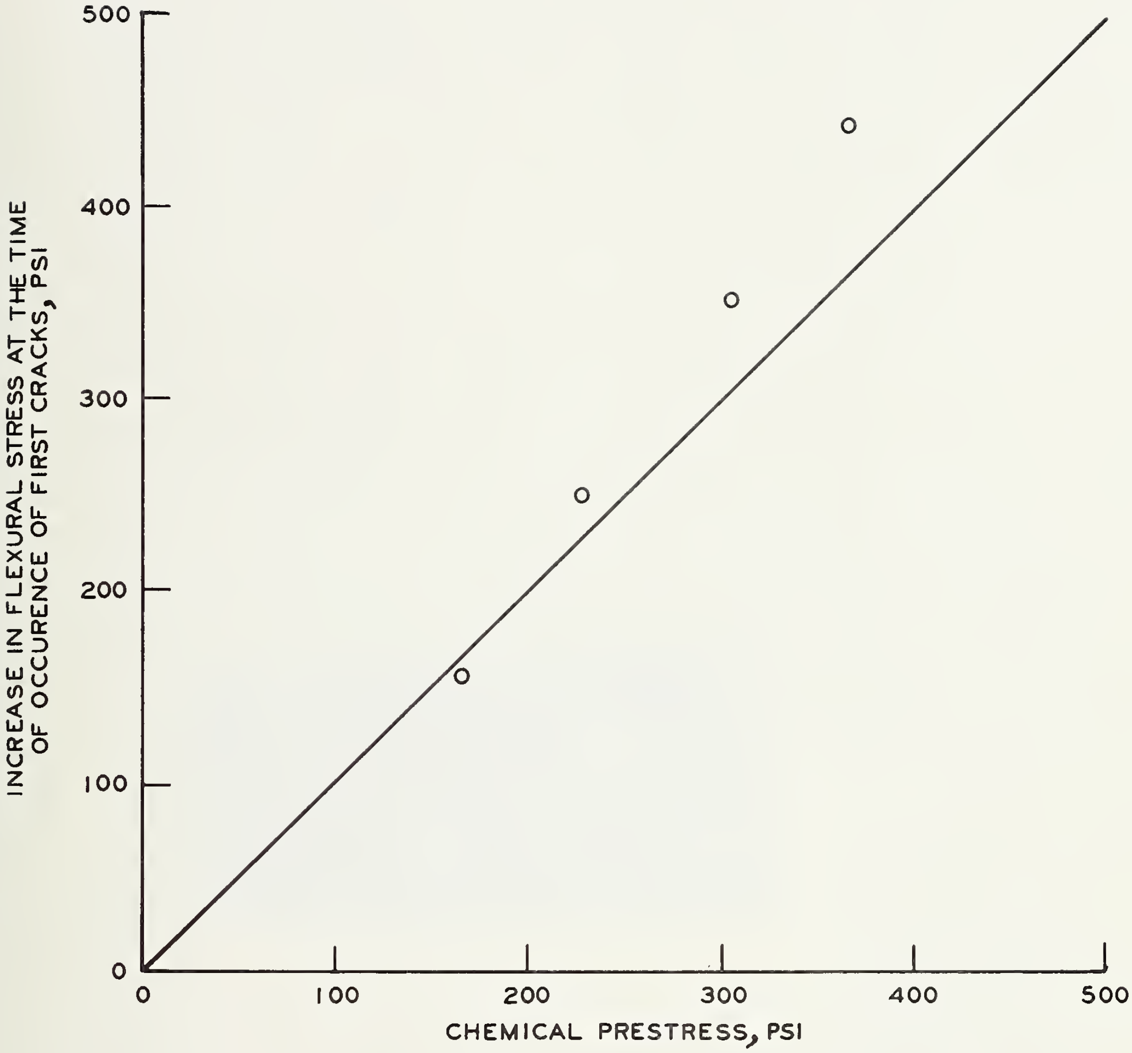

Figure 8.16. Relation between chemical prestress and amount of flexural stress increase at the time of occurence of cracks*

*See page 210 for reference.

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