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16. Abstract

This study involves an evaluation of the blackbase mixture design procedure currently being used by the Texas State Department of Highways and Public Transportation (DHT). All specimens were prepared and tested according to current Texas DHT standards. The evaluation involved testing blackbase specimens using the static and repeated-load indirect tensile tests to determine certain engineering properties, i.e., tensile strength, static modulus of elasticity, fatigue life, resilient modulus of elasticity, and resistance to permanent deformation, for various blackbase mixtures. Generally, it was found that these properties were maximum at asphalt contents less than the asphalt contents determined by the Texas method of design for blackbase mixtures. Thus, it is recommended that the static and repeated-load indirect tensile test should be conducted as a part of the mixture design procedure.

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by

David B. Peters
Thomas W. Kennedy

Research Report Number 183-11

Tensile Characterization of Highway Pavement Materials
Research Project 3-9-72-183

conducted for

Texas
State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

March 1979

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This is the eleventh in a series of reports dealing with the findings of a research project concerned with tensile and elastic characteristics of highway pavement materials. This report summarizes the results of a limited study to evaluate the blackbase design procedures currently used by the Texas State Department of Highways and Public Transportation. The evaluation was based upon the results obtained using the static and repeated-load indirect tensile tests on various blackbase mixtures with asphalt contents above and below those determined using the current blackbase design method.

This report was completed with the assistance of many people. Special appreciation is due James N. Anagnos and Pat S. Hardeman for their guidance and assistance in the testing program and Frank E. Herbert, Gerald Peck, and Robert E. Long, of the Texas State Department of Highways and Public Transportation, who provided technical liason and support for the project. Appreciation is also extended to personnel from Districts 5, 11, and 13 who assisted in obtaining the blackbase materials used on the project and to the Center for Highway Research staff who assisted in the preparation of the manuscript. The support of the Federal Highway Administration, Department of Transportation, is gratefully acknowledged.

LIST OF REPORTS

Report No. 183-1, "Tensile and Elastic Characteristics of Pavement Materials," by Bryant P. Marshall and Thomas W. Kennedy, summarizes the results of a study on the magnitude of the tensile and elastic properties of highway pavement materials and the variations associated with these properties which might be expected in an actual roadway.

Report No. 183-2, "Fatigue and Repeated-Load Elastic Characteristics of Inservice Asphalt-Treated Materials," by Domingo Navarro and Thomas W. Kennedy, summarizes the results of a study on the fatigue response of highway pavement materials and the variation in fatigue life that might be expected in an actual roadway.

Report No. 183-3, "Cumulative Damage of Asphalt Materials Under Repeated-Load Indirect Tension," by Calvin E. Cowher and Thomas W. Kennedy, summarizes the results of study on the applicability of a linear damage rule, Miner's Hypothesis, to fatigue data obtained utilizing the repeated-load indirect tensile test.

Report No. 183-4, "Comparison of Fatigue Test Methods for Asphalt Materials," by Bryon W. Porter and Thomas W. Kennedy, summarizes the results of a study comparing fatigue results of the repeated-load indirect tensile test with the results from other commonly used tests and a study comparing creep and fatigue deformations.

Report No. 183-5, "Fatigue and Resilient Characteristics of Asphalt Mixtures by Repeated-Load Indirect Tensile Test," by Adedare S. Adedimila and Thomas W. Kennedy, summarizes the results of a study on the fatigue behavior and the effects of repeated tensile stresses on the resilient characteristics of asphalt mixtures utilizing the repeated-load indirect tensile test.

Report No. 183-6, "Evaluation of the Resilient Elastic Characteristics of Asphalt Mixtures Using the Indirect Tensile Test," by Guillermo Gonzalez, Thomas W. Kennedy, and James N. Anagnos, summarizes the results of a study to evaluate possible test methods for obtaining elastic properties of pavement materials, to recommend a test method and preliminary procedure, and to evaluate properties in terms of mixture design.

Report No. 7, "Permanent Deformation Characteristics of Asphalt Mixtures by Repeated-Load Indirect Tensile Test," by Joaquin Vallejo, Thomas W. Kennedy, and Ralph Haas, summarizes the results of a preliminary study which compared and evaluated permanent strain characteristics of asphalt mixtures using the repeated-load indirect tensile test.

Report No. 183-8, "Resilient and Fatigue Characteristics of Asphalt Mixtures Processed by the Dryer-Drum Mixer," by Manuel Rodriguez and Thomas W. Kennedy, summarizes the results of a study to evaluate the engineering properties of asphalt mixtures produced using a dryer-drum plant.

Report No. 183-9, "Fatigue and Repeated-Load Elastic Characteristics of Inservice Portland Cement Concrete," by John A. Crumley and Thomas W. Kennedy, summarizes the results of an investigation of the resilient elastic and fatigue behavior of inservice concrete from pavements in Texas.

Report No. 183-10, "Development of a Mixture Design Procedure for Recycled Asphalt Mixtures," by Ignacio Perez, Thomas W. Kennedy, and Adedare S. Adedimila, summarizes the results of a study to evaluate the fatigue and elastic characteristics of recycled asphalt materials and to develop a preliminary mixture design procedure.

Report No. 183-11, "An Evaluation of the Texas Blackbase Mix Design Procedure Using the Indirect Tensile Test", by David B. Peters and Thomas W. Kennedy, summarizes the results of a study evaluating the elastic and repeated-load properties of blackbase mixes determined from current blackbase design procedures using the indirect tensile test.

ABSTRACT

This study involves an evaluation of the blackbase mixture design procedure currently being used by the Texas State Department of Highways and Public Transportation (DHT). All specimens were prepared and tested according to current Texas DHT standards. The evaluation involved testing blackbase specimens using the static and the repeated-load indirect tensile tests to determine certain engineering properties, i.e., tensile strength, static modulus of elasticity, fatigue life, resilient modulus of elasticity, and resistance to permanent deformation, for various blackbase mixtures. Generally, it was found that these properties were maximum at asphalt contents less than the asphalt contents determined by the Texas method of design for blackbase mixtures. Thus, it is recommended that the static and repeated-load indirect tensile test should be conducted as a part of the mixture design procedure.

Key Words: blackbase, mixture design, static and repeated-load indirect tensile tests, engineering properties, tensile strength, static modulus, resilient modulus, fatigue life, permanent deformation.

SUMMARY

The purpose of this study was to evaluate the blackbase mixture design procedure used by the Texas State Department of Highways and Public Transportation. The evaluation was based upon a comparison of the engineering properties obtained using the static and the repeated-load indirect tensile on mixtures with various asphalt contents with the results for mixtures designed using the current blackbase design procedure.

For this study three blackbase mixtures, one with a crushed gravel and field sand, one with a crushed limestone, and one with a field sand, were used. Each of these mixtures has been used in a blackbase mixture. Optimum asphalt contents designed by the Texas method were compared to the optimum asphalt contents determined for specific engineering properties, i.e., tensile strength, static modulus of elasticity, fatigue life, resilient modulus of elasticity, and resistance to permanent deformation. All of the engineering properties were determined at 10, 24, and 38°C (50, 75, and 100°F).

Generally, the results indicate that the optimum asphalt contents for the various engineering properties obtained using the static and repeated-load indirect tensile tests were less than the optimum for the Texas method of blackbase design. It also appeared that Test Method Tex-126-E did not always identify the field performance of blackbase mixtures. As a result, it was recommended that static and repeated-load tests be performed as part of the blackbase procedure in order to determine the design.

IMPLEMENTATION STATEMENT

Based on the findings of this study and other studies it is recommended that the static and repeated-load indirect tensile tests be used to evaluate the engineering properties of blackbase mixtures. In addition, it is recommended that these test methods be conducted as part of the blackbase mixture design procedure since it was generally found that the engineering properties were maximum at asphalt contents less than the optimum asphalt content determined when the current design procedure is used. This would allow the engineering properties to be used as a criterion of design and would provide a basis for possible mixture design modifications.

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CHAPTER 1. INTRODUCTION

The purpose of the investigation summarized herein was to evaluate the hot-mix blackbase mixture design method used by the Texas State Department of Highways and Public Transportation (DHT).

Usually, hot-mix mixture design methods have involved the evaluation of properties such as stability, density, percent asphalt, and air voids. The Texas method is used only for the design of blackbase mixtures, which contain larger aggregates than asphalt surface mixtures, and it uses the relationship between total air voids and asphalt content for determining a design optimum asphalt content. The acceptability of the mixture containing this design asphalt content is determined by whether or not it satisfies unconfined compressive strength requirements established in Test Method Tex-126-E (Ref 1).

The DHT generally has found Test Method Tex-126-E to be satisfactory in providing serviceable bases for pavements. However, recent usage of the Texas method has often resulted in changing the design asphalt content obtained by Test Method Tex-126-E, depending on what recent field performances with that material or similar materials have indicated. Another problem which has been observed in using the Texas method is that the unconfined strength requirements outlined in the test method do not suitably identify pavement materials having good or poor field performances.

TEXAS BLACKBASE DESIGN PROCEDURE

The blackbase mix design procedure used in Texas was developed in about 1970 and is described in Test Method Tex-126-E (Refs 1 and 17), which outlines the procedure for selecting the optimum asphalt content for both laboratory and field mixtures of blackbase which have maximum aggregate size of up to 45 mm (1-3/4 in.).

Test Method Tex-126-E includes procedures for the following seven phases of the mix design:

(1) preparing materials,

(2) weight batching of materials to be mixed,

(3) determining specific gravity of aggregates,

(4) mixing and molding blackbase specimens and determining percent total air voids,

(5) pressure wetting of blackbase specimens,

(6) testing blackbase specimens in unconfined compression, and

(7) determining field density, percent total voids, percent asphalt, minimum allowable density, and actual percent density for field control.

The basic procedure begins in the laboratory and involves mixing and compacting an aggregate with various quantities of asphalt and then determining the percent total air voids at each asphalt content. An asphalt content-air voids relationship, or AVR curve, is then developed (Fig 1). The AVR design optimum asphalt content is slightly greater than the asphalt content corresponding to the inflection point of the straight line section of the laboratory AVR curve (Fig 1). Specimens of the mixture containing this design asphalt content are prepared and unconfined compression tests are performed to classify the mixture and determine its acceptability according to specified strength values.

To allow for changes in gradation of stockpiled aggregate, the actual plant mixture which has been through the batch mixer must be evaluated. Before laboratory compaction of specimens containing stockpiled aggregate, various amounts of asphalt are added in order to develop a field AVR curve. The field AVR curve is constructed by drawing a line which is parallel to the left leg of the laboratory AVR curve and which passes through the lean field mix point. A curve is drawn through the richer field mix points, parallel to the richer portion of the laboratory AVR curve. The intersection of the richer, curved, line and the lean, straight, line is the field design optimum asphalt content for the field mixture.

PAVEMENT DISTRESS

Several basic distress modes have been defined for asphalt pavements. Four of these distress modes are relevant to base layers: (1) thermal or shrinkage cracking, (2) fatigue cracking, (3) permanent deformation, or rutting, and (4) disintegration, e.g., stripping, raveling, etc. Therefore, mixture design and evaluation of pavement materials should consider these

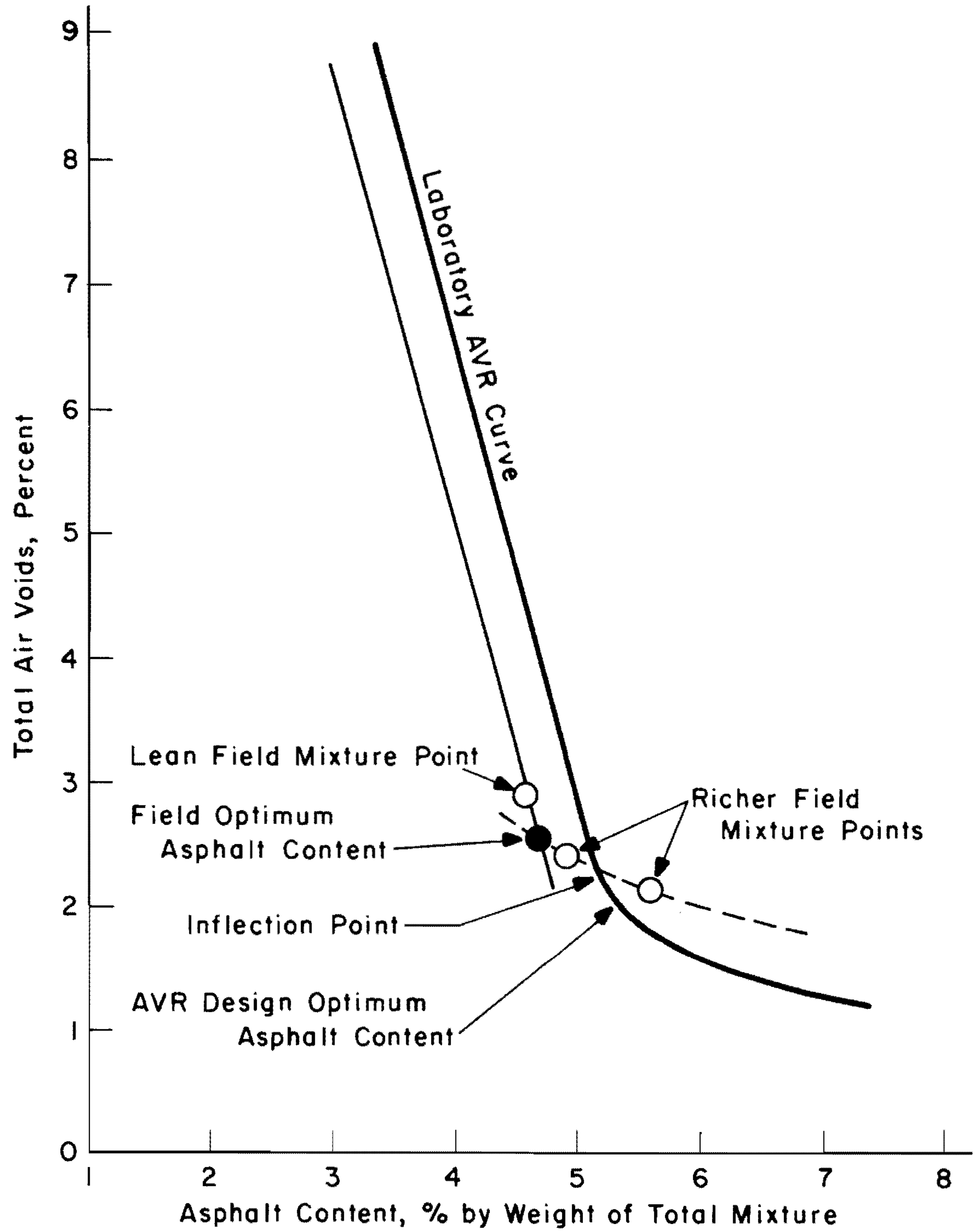


Fig 1. Relationship between asphalt content and total air voids (Ref 1).

distress modes and the fundamental engineering properties related to these forms of distress.

The current mixture design method used by the DHT does not contain design criteria involving those engineering properties related to distress. This investigation, therefore, was designed to evaluate the Texas method in terms of tensile strength and elastic, fatigue, and permanent deformation properties of the blackbase mixtures at several asphalt contents. The properties at the various asphalt contents were compared to the properties at the design optimum asphalt content obtained by the Texas method.

PROPOSED TEST METHOD

The tests selected for evaluating the Texas method of mix design were the static and repeated-load indirect tensile tests, which were developed at the Center for Highway Research and are being used by other agencies. Both forms of the indirect tensile test measure the tensile properties of pavement materials which directly relate to the common mode of structural failure, or tension, and provides information on tensile strength, modulus of elasticity, and Poisson's ratio for both static and repeated loads, fatigue characteristics, and permanent deformation characteristics of pavement materials. In addition, the test has the following characteristics:

(1) The test is relatively simple to conduct.

(2) Either cylindrical laboratory specimens or cores can be used.

(3) The type of specimen and the equipment are the same as that used for compression testing.

(4) Failure is not seriously affected by surface conditions of the specimen.

(5) Failure is initiated in a region of relatively uniform tensile stress.

(6) The coefficient of variation of test results is low compared to that of other test methods.

A comprehensive description of the experimental program, materials, and procedures used in this investigation is contained in Chapter 2. The analysis and the discussion of test results are presented in Chapter 3 and the conclusions and recommendations based on the findings of this study are listed in Chapter 4.

CHAPTER 2. EXPERIMENTAL PROGRAM

The purpose of this investigation was to evaluate the blackbase design procedure currently used by the Texas State Department of Highways and Public Transportation (Test Method Tex-126-E, Refs 1 and 17).

The basic approach was to compare the engineering properties of blackbase mixtures at various asphalt contents with the properties of mixtures at the design asphalt content obtained using the current Texas design procedure. The engineering properties were determined at 10, 24, and 38°C (50, 75, and 100°F). Three asphalt mixtures currently used in the construction of actual pavements by the Texas State Department of Highways and Public Transportation (DHT) were tested using the static and the repeated-load indirect tensile tests and the unconfined compression test.

This chapter describes the materials, testing equipment, testing procedures, and experiment design used in the investigation.

MATERIALS

The three aggregates used in this investigation were obtained from Eagle Lake, Lubbock, and Lufkin, Texas. Each of these aggregates has been used in pavements and has performed satisfactorily; however, when mixtures containing these aggregates were tested in unconfined compression by the DHT according to Test Method Tex-126-E, only the Lubbock mixture satisfied the specified strength requirements.

Eagle Lake Material

The Eagle Lake aggregate is a mixture of four different aggregates combined in the following proportions:

Aggregate	Percent
Lone Star coarse aggregate	43
Tanner Walker sand	35
Lone Star Gem sand	12
Stiles coarse sand	10
	100

The gradations of the individual aggregates in the Eagle Lake gravel are shown in Appendix B. Lone Star Gem sand and Lone Star coarse aggregate are siliceous river gravels with crushed faces. Tanner Walker sand and Stiles coarse sand are field sands. The combination of these aggregates can be generally described as a smooth angular non-porous crushed river gravel. The resulting gradation is shown in Fig 2 and listed in Appendix A.

The asphalt cement which was mixed with the Eagle Lake aggregate is the same as that used with the Eagle Lake aggregate for blackbase construction. The asphalt cement was an AC-20 produced at the Exxon refinery in Baytown, Texas, and supplied by the Yoakum District. The asphalt properties, as determined by the Texas State Department of Highways and Public Transportation, are summarized in Table 1.

Lubbock Material

Lubbock aggregate is a rough sub-angular, porous, crushed limestone (caliche). The source is Long Pit, located approximately ten miles southeast of Lubbock. This aggregate was used for the blackbase construction of I-27 between the North Loop of Lubbock and New Deal, Texas. The washed aggregate gradation which was used in the construction of I-27 is shown in Fig 2 and is listed in Appendix A.

The asphalt cement was an AC-10 produced by the Cosden Oil refinery in Big Spring, Texas. The asphalt properties as determined by the DHT are summarized in Table 1.

Lufkin Material

Lufkin aggregate is a combination of two pit sands, both excavated from Seal Pit. The two sands, one slightly finer than the other, are mixed in equal proportions to obtain the desired gradation. The combined aggregate gradation is shown in Fig 2 and is listed in Appendix A. This aggregate was

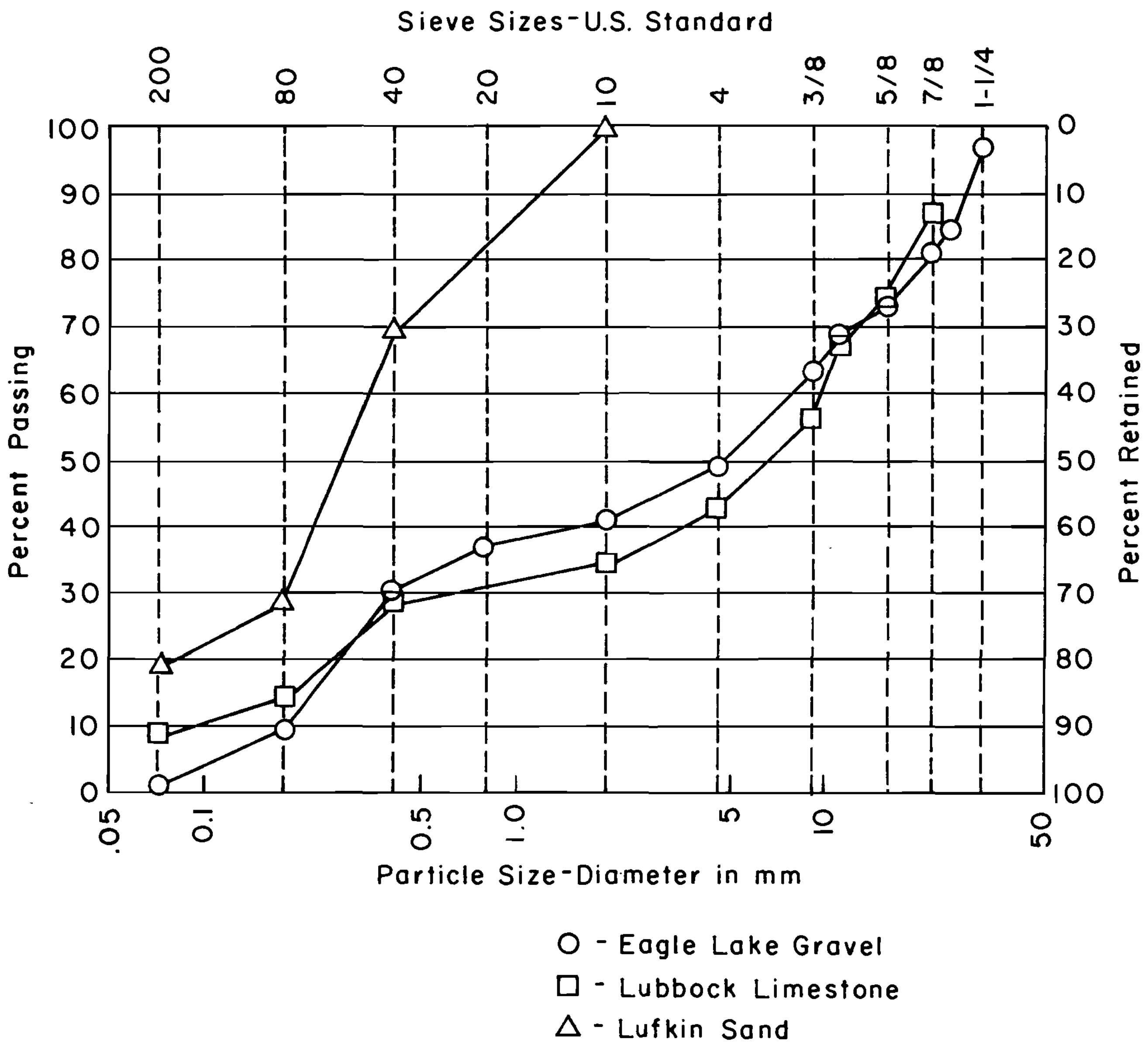


Fig 2. Washed gradations for Eagle Lake gravel, Lubbock limestone, and Lufkin sand aggregates.

TABLE 1. SUMMARY OF PROPERTIES OF ASPHALT CEMENTS USED*

	Eagle Lake	Lubbock	Lufkin
Asphalt type	AC-20	AC-10	AC-20
Producer	Exxon	Cosden Oil	Texaco
Water, percent	nil	nil	nil
Viscosity at 135°C (275°F), stokes	3.3	2.5	4.73
Viscosity at 60°C (140°F), stokes	2,093	912	1,876
Solubility in CCl_4, percent	>99.7	>99.7	—
Flash point, C.O.C., °C (°F)	>315 (600)	>315 (600)	310 (590)
Ductility at 25°C (77°F), 5 cm/min, cm	—	—	—
Penetration at 25°C (77°F), 100 g, 5 sec	56	86	84
Specific gravity at 25°C (77°F)	1.020	1.026	1.020
Tests on residues from thin film oven test:			
Viscosity at 60°C (140°F), stokes	3,574	2,172	3,956
Ductility at 25°C (77°F), 5 cm/min, cm	>141	>141	>141
Spot test	neg	neg	—

*as reported by the Department of Highways and Transportation used for the blackbase construction of SH 121 in Angelina County, Texas. The gradation of the individual sands is shown in Appendix B.

The asphalt cement was an AC-20 produced at the Texaco refinery in Port Neches, Texas. The properties of the asphalt cement are summarized in Table 1.

SPECIMEN PREPARATION

All specimens prepared for this investigation were mixed and compacted according to Test Method Tex-126-E except that the mixing was done using an 11-liter (12-quart) capacity Hobart mixer rather than by hand. The complete mixing and compaction procedures are given in Appendix C and are summarized below.

The aggregates were batched by dry weight, mixed at 177°C (350°F), and compacted at 127°C (260°F). Compaction was performed using the Texas gyratory-shear compactor. The maximum compressive stress, 3450 kPa (500 psi), was applied to the specimens after gyration. This stress was maintained until the vertical deformation rate was less than 0.13 mm (0.005 in.) per 5-minute period, at which time the in-mold AVR density was determined. The resulting specimens were approximately 152 mm (6 in.) in diameter and 200 mm (8 in.) in height.

Immediately after the AVR density determinations were made, the specimens for the unconfined compression tests were extruded from the mold and cured overnight at 60°C (140°F), according to Test Method Tex-126-E.

The specimens for the static and repeated-load indirect tensile tests were allowed to cool in the compaction mold for about one hour before being extruded. This was done to prevent slumping of the specimen, which if it occurred would result in a nonuniform diameter. After removal from the mold, the specimens were allowed to cure overnight at room temperature. Specimens were cut from the top portion and bottom portion of the original specimen. The densities of these specimens were measured to determine whether there were differences and to obtain the densities of the specimens being tested. The top and bottom specimens were then cured overnight at the designated testing temperature; thus, the total curing time for the indirect tensile specimens was two days.

The specimens for the unconfined compression tests were about 200 mm (8 in.) in height and 152 mm (6 in.) in diameter. The specimens cut for the indirect tensile tests were generally 152 mm (6 in.) in diameter and about 84 mm (3.3 in.) in height. The specimens used in the repeated-load tests for the Eagle Lake gravel, however, were 152 mm (6 in.) in diameter but varied from 51 to 102 mm (2 to 4 in.) in height. This variation in height was necessary because of loading restrictions in the pneumatic repeated-load system.

TESTING EQUIPMENT

Three basic types of tests were conducted, the unconfined compression test and the static and repeated-load indirect tensile tests. The basic equipment used for these tests is described below.

Unconfined Compression Tests

The testing equipment for the unconfined compression tests included the Rainhart pressure pycnometer and the Texas gyratory-shear compaction device. The pressure pycnometer was used to subject the specimens to an 8300-kPa (1200-psi) hydrostatic water pressure at a water temperature of 65°C (150°F) for 15 minutes prior to actual testing (Test Method Tex-109-E, Part IV). All unconfined compression tests were conducted using as a testing device the Texas gyratory-shear compactor, which is capable of applying and maintaining loads of approximately 89 kN (20,000 lb) on a 152-mm (6-in.)-diameter specimen for deformation tests of up to 254 mm (10 in.) per minute.

Static Indirect Tensile Tests

The testing equipment for the static indirect tensile tests was the same as that reported in previous Center for Highway Research investigations (Refs 2 and 3). The basic testing apparatus was an MTS closed-loop electrohydraulic loading system. The vertical deformations were measured by a DC linear variable differential transducer. Horizontal deformations were measured by two cantilevered arms wired with strain gages. The load-horizontal and load-vertical deformations were recorded on a pair of X-Y plotters, Hewlett Packard Models 7001A and 7000AR.

Repeated-Load Indirect Tensile Tests

For the repeated-load tests, two loading systems were used. One, the MTS electrohydraulic loading system described above, was used for the repeated-load tests of the Lubbock limestone and the Lufkin sand mixtures. Because of the extensive amount of time required to conduct repeated-load tests, a pneumatic system was developed and used for the repeated-load tests on the Eagle Lake gravel.

The pneumatic system (Fig 3) was driven by a source pressure of 620 kPa (90 psi) with the load controlled by a regulator. The load was transferred to the specimen by means of a diaphragm type air piston, characterized by low frictional losses. A triangular cam, rotating at 20 revolutions per minute, actuated a microswitch once every second which actuated a solenoid controlling the air flow to the air piston. An adjusting screw produced a load-time pulse which was identical to the load-time pulse of the MTS system.

The horizontal and vertical deformations were measured by DC linear variable differential transducers and recorded on the Hewlett Packard X-Y plotters. Typical traces are shown in Fig 4.

TESTING PROCEDURE

Unconfined Compression Tests

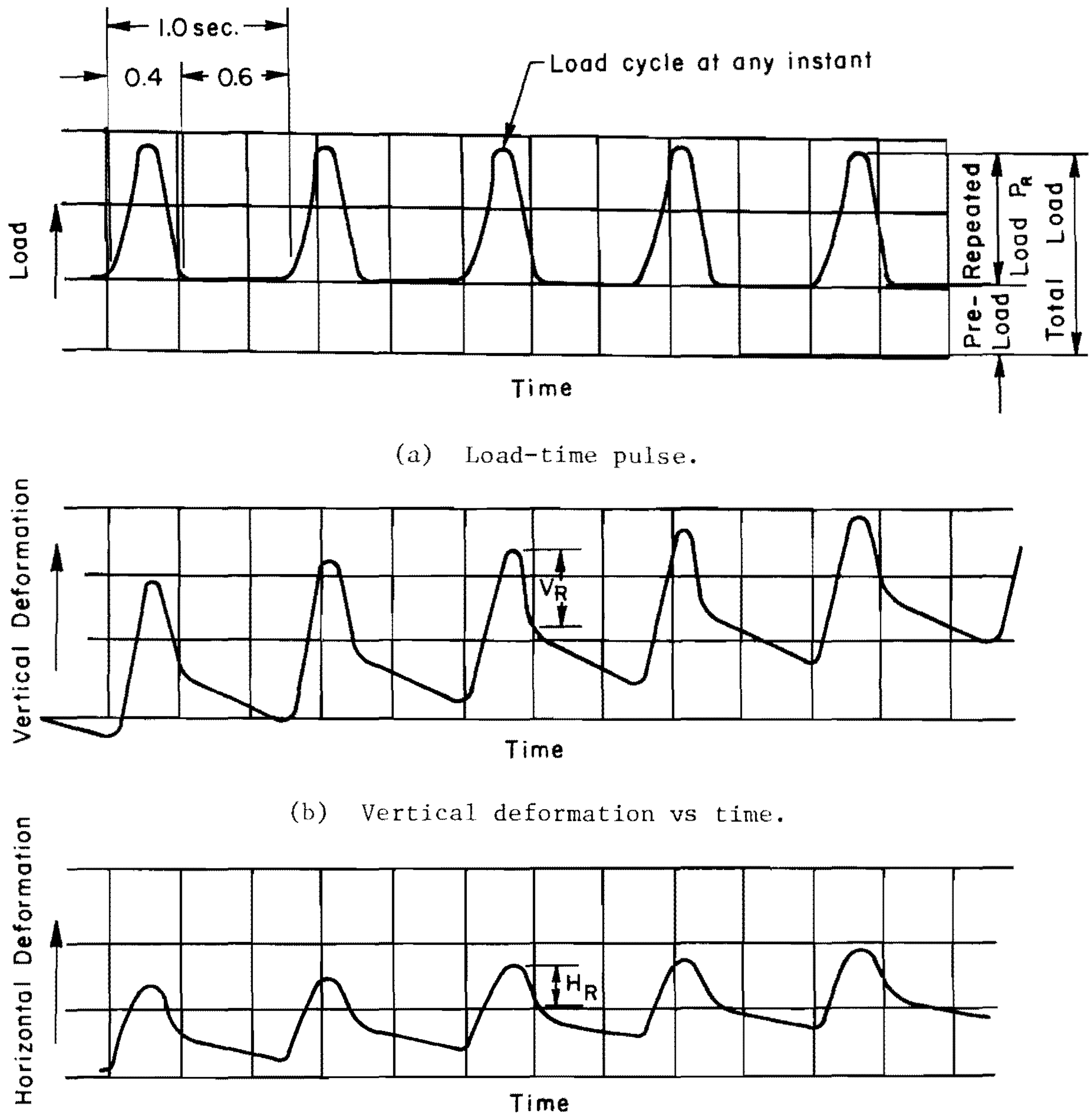
After compaction, the specimens were cured overnight at 60°C (140°F). The specimens were then pressure wetted by subjecting the specimens to a hydrostatic water pressure of 8300 kPa (1200 psi) at 65°C (150°F) for 15 minutes (Test Method Tex-109-E, Part IV). Immediately after the pressure wetting, the specimens were tested in unconfined compression. Duplicate specimens were tested at two different deformation rates. One was tested at a fast rate of deformation, 245 mm (10 in.) per minute, and the other at a slow rate, 3.8 mm (0.15 in.) per minute. The maximum load attained was recorded and used to calculate the unconfined compressive strength. These strength values were then compared to Test Method Tex-126-E specifications to determine whether the mixture was satisfactory (Table 2).

Static Indirect Tensile Tests

A preload of 90 N (20 lb), which produced a tensile stress of approximately 4 kPa (0.6 psi), was applied to the specimens in the static tests to prevent an impact loading and to minimize the effect of seating of the loading strip. The specimens were then loaded at a constant deformation rate of 51 mm (2 in.) per minute.

The load-vertical deformation and load-horizontal deformation relationships were recorded by a pair of X-Y plotters. The tensile strength was calculated using the ultimate load carried by the specimen rather than the first

Fig 3. Pneumatic system loading apparatus.



(a) Load-time pulse.

(b) Vertical deformation vs time.

(c) Horizontal deformation vs time.

Fig 4. Typical load pulse and the corresponding relationships between deformations and time for repeated-load indirect tensile test.

TABLE 2. MINIMUM UNCONFINED COMPRESSIVE STRENGTHS FOR VARIOUS GRADES OF BLACKBASE AS REQUIRED BY TEST METHOD TEX-126-E

Grade No.	Slow Strength,* kPa (psi)	Fast Strength,** kPa (psi)
1	345 (50)	690 (100)
2	276 (40)	690 (100)
3	207 (30)	690 (100)

*Slow speed = 3.8 mm/min (0.15 in./min)

**Fast speed = 254 mm/min (10.0 in./min) inflection point on the load-deformation relationships, which was done in some of the previous studies, since a first inflection point was not readily identified in many of the tests.

Repeated-Load Indirect Tensile Tests

For the repeated-load tests, also, a preload was applied. The preload for the pneumatic system was 140 N (32 lb) and was equal to the weight of the platen. A preload of 90 N (20 lb) was used for the MTS system. The difference in the preload would be expected to produce differences in the measured engineering properties. Future testing should eliminate any difference in preload and should minimize the magnitude of the preload.

The desired load was applied at a frequency of one cycle per second (1 Hz) with a 0.4-second load duration and a 0.6-second rest period. A typical load pulse and the resulting deformation relationships are shown in Figs 4 and 5.

PROPERTIES

The properties and characteristics of the asphalt mixtures which were analyzed were

(1) tensile strength,
(2) static Poisson's ratio,
(3) static modulus of elasticity,
(4) fatigue life,
(5) resilient Poisson's ratio,
(6) resilient modulus of elasticity,
(7) permanent deformation,
(8) AVR density, and
(9) total air voids.

Several of these properties are directly or indirectly related to the relevant pavement distress modes previously discussed. These properties and the equations (Refs 16 and 17) used to calculate these properties are discussed below.

Tensile Strength

The ultimate tensile strength is a measure of the maximum stress which the mixture can withstand and is related to the mixture's resistance to thermal and shrinkage cracking.

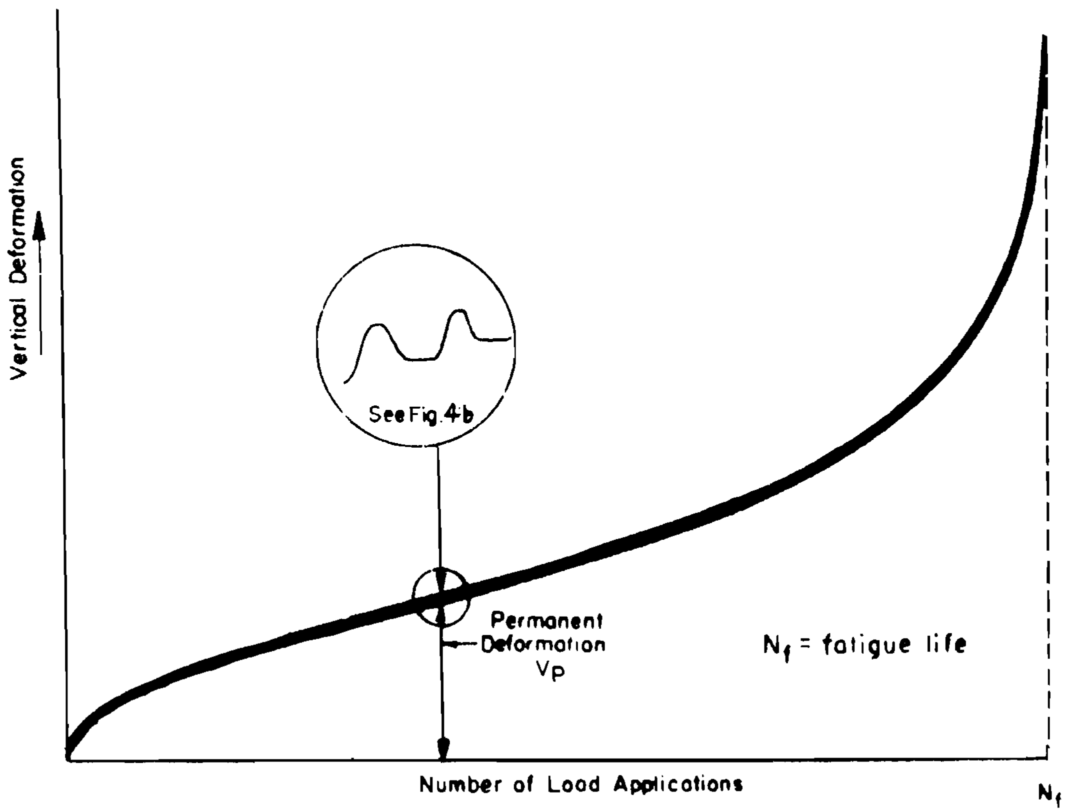


Fig 5. Relationship between number of load applications and vertical deformation for the repeated-load indirect tensile test.

The ultimate tensile strength was calculated using the following relationship for 152-mm (6-in.)-diameter specimens and the load-deformation information obtained from the static indirect tensile test:

$$S_T = \frac{0.105\ P_{ult}}{t}$$

where

S_T = ultimate tensile strength, psi,

P_{ult} = maximum load carried by the specimen, lb, and

t = thickness or height of the specimen, in.

Tensile stresses produced by loads less than the maximum load P_{ult} can also be calculated using the above equation.

Static Poisson's Ratio

The static Poisson's ratio was calculated from the relationship between the vertical and horizontal deformations, which was generally linear, up to a sharp inflection point that generally occurred between 60 and 90 percent of the ultimate load. If a sharp break in the curve was not present, data points were included up to a point about midway between the ultimate load and the deviation from linearity. The equation used to determine static Poisson's ratio was

$$\nu = \frac{4.09}{DR} - 0.27$$

where

ν = static Poisson's ratio and

DR = deformation ratio, the slope of the relationship between vertical deformation and horizontal deformation, inches of vertical deformation per inch of horizontal deformation.

Static Modulus of Elasticity

The static modulus of elasticity was determined by analyzing the load-deformation relationships for static tensile tests. A regression analysis was conducted on data points up to the sharp inflection point in the load-deformation curves or to about the midpoint if a sharp inflection point was not present. This is basically the same procedure as that suggested by Anagnos and Kennedy (Refs 2 and 16).

The equation used to calculate the static modulus of elasticity was

$$E_s = \frac{S_h}{t} (0.27 + \nu)$$

where

E_s = static modulus of elasticity, psi, and

S_h = the slope of the relationship between load and horizontal deformation, lb/in.

Fatigue Life

Fatigue life is defined as the number of load applications at which the specimen will no longer resist load or at which deformation is excessive and increases with essentially no additional loads (Fig 5).

Resilient Poisson's Ratio

The resilient Poisson's ratio ν_R was determined from the repeated-load tests and calculated using the resilient vertical and horizontal deformations (Fig 4) V_R and H_R for the loading cycle corresponding to $0.5\ N_f$. The equation is the same as that used for the static Poisson's ratio; however, since the relationships between load and deformation are essentially linear, the equation has been modified and expressed as follows:

$$\nu_R = 4.09 \frac{H_R}{V_R} - 0.27$$

where

H_R and V_R are the resilient horizontal and vertical deformations as shown in Fig 4.

The values of resilient Poisson's ratio which were used to calculate the resilient modulus of elasticity are not discussed in this report but are listed in Appendix F.

Resilient Modulus of Elasticity

The resilient modulus of elasticity is calculated by using the resilient, or recoverable, horizontal and vertical deformations, which are more characteristic of the elastic deformations produced by rapidly applied, repeated loads. The equation used to calculate the resilient modulus is

$$E_R = \frac{P}{t\, H_R} (0.27 + \nu_R)$$

where

E_R = resilient modulus of elasticity, psi, and

P = the applied load, lb (Fig 4).

Permanent Deformation

The parameter selected for evaluation of resistance to permanent deformation was permanent vertical deformation per cycle, which is the slope of the relationship between permanent vertical deformation and number of load applications. The slope was determined by least squares regression for the portion of the relationship between $0.10\ N_f$ and $0.70\ N_f$, which is essentially linear (Fig 5). Several other permanent deformation characteristics, e.g., average permanent deformation and initial permanent deformation, were investigated and found to be of little value.

For the purpose of permanent deformation prediction for the field, permanent strain would be more useful than permanent vertical deformation per cycle. Permanent strain was not used for this analysis because permanent horizontal deformations were not measured in the repeated-load tests. Therefore, Poisson's ratio for cumulative permanent deformation could not be obtained.

AVR Density

The AVR density was calculated using the mold diameter and the measured height, which was obtained while the specimen was subjected to the final compaction load of 3450 kPa (500 psi). This is also referred to as the in-mold AVR density and is used to calculate percent total air voids as defined by Test Method Tex-126-E. The weight of the specimen was determined after it was extruded from the mold. The AVR density was determined according to the following equation:

$$\text{AVR density} = \frac{W}{H \frac{\pi D^2}{4}}$$

where

AVR density = unit weight of compacted specimen, pcf,

W = weight of specimen, lb,

H = height of specimen in mold while subjected to final compaction pressure of 3450 kPa (500 psi), ft, and

D = diameter of mold, ft.

Total Air Voids

To obtain the percent total air voids, the following value was determined, as specified by Test Method Tex-126-E:

$$\text{Zero air void density (ZAVD)} = \frac{100\,\gamma_w}{\frac{P_s}{G_s} + \frac{P_a}{G_a}}$$

where

γ_w = unit weight of water,

P_s = percent dried aggregate by weight of the total mixture,

P_a = percent asphalt by weight of the total mixture,

G_s = absolute specific gravity of the combined aggregate (obtained by performing Test Method Tex-109-E, Part IV, using the pressure pycnometer), and

G_a = absolute specific gravity of the asphalt (from asphalt tests, Table 1).

The percent total air voids was determined from the following relationship:

$$\text{Percent total air voids} = 1 - \frac{\text{AVR density of specimen}}{\text{ZAVD}} \times 100$$

EXPERIMENTAL DESIGN

The levels of each factor and the number of specimens at each level were selected through a step-by-step process to conserve time, money, and materials. A full factorial design was not used; rather, the various cells were selectively chosen so that optimum asphalt contents for the various engineering properties studied could be found using a minimum number of specimens. These tests were performed at 10, 24, and 38°C (50, 75, and 100°F) in controlled environment chambers and at different stress levels for the repeated-load tests. For the repeated-load tests, two stress levels (Table 3) which would produce reasonable fatigue lives were selected. A summary of the testing program of this investigation is shown in Fig 6.

TABLE 3. STRESS LEVELS FOR REPEATED-LOAD INDIRECT TENSILE TESTS

Mixtures	Temperature, °C(°F)	Stress Level, kPa (psi) Low σ_L	Stress Level, kPa (psi) High σ_H
Eagle Lake	10 (50)	215 (31.2)	—
	24 (75)	40 (5.8)	120 (17.4)
	38 (100)	25 (3.6)	40 (5.3)
Lubbock limestone	10 (50)	550 (79.8)	1000 (145)
	24 (75)	150 (21.7)	250 (36.2)
	38 (100)	80 (11.8)	120 (17.4)
Lufkin sand	10 (50)	280 (40.6)	600 (87.0)
	24 (75)	60 (8.7)	180 (26.1)
	38 (100)	20 (2.9)	40 (4.8)

Temperature	Type of Test	Stress Level	Eagle Lake Gravel-AC-20					Lubbock Limestone-AC-20							Lufkin Sand-AC-20							
		Asphalt Content, %	3.5	4.0	4.5	5.0	5.5	5.5	6.0	6.5	7.0	7.5	8.0	8.5	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10°C	T_S		2	2	2	2				4	5	5	2	2	2	2	2	2	2			
(50°F)	T_R	σ_H								2	2	3	2			1	2	2	2	1		
		σ_L	1	2	3	3				1	2	3	2			1	2	2	2	1		
24°C	U			2*	2*	2*	2*				3*	2*	3*	2*	1	2*	2*	2*	2*	2*	2*	
	T_S		2	2	2	2			2	4	6	4	3		2	3	3	3	1			
(75°F)	T_R	σ_H	2	2	2	2				1	3	3	2			2	2	2	2			
		σ_L	3	3	2	2				1	3	3	2			2	2	2	2			
38°C	T_S		2	2	2	2		2	4	4	4	2	2		2	3	3	3	1			
	T_R	σ_H	2	2	2	2				1	2	2	2			1	2	3	3	2	1	1
(100°F)		σ_L	1	2	2	2				1	2	2	2			1	2	3	3	4	1	1

U = unconfined compression test

T_S = static indirect tensile test

T_R = repeated-load indirect tensile test

σ_L = repeated-load indirect tensile test at the low stress level in Table 2

σ_H = repeated-load indirect tensile test at the high stress level in Table 2

* = one specimen was tested at high loading rate and either one or two at low loading rate

Fig 6. Graphical representation of experimental tests.

CHAPTER 3. ANALYSIS AND DISCUSSION OF TEST RESULTS

The basic approach used to evaluate the Texas method of blackbase mixture design was to compare the various engineering properties for a range of asphalt contents with the engineering properties at the AVR design optimum asphalt content.

DESIGN ASPHALT CONTENTS

A laboratory design asphalt content, or an AVR design optimum, was determined for each material from the relationship between asphalt content and total air voids. Total air voids were calculated using the in-mold AVR density and zero air void density as described in Chapter 2. The AVR design optimum asphalt content was chosen slightly greater than the asphalt content corresponding to the inflection point on the straight line section of the AVR curve (Fig 1).

Eagle Lake Gravel Mixtures

The relationship between asphalt content and total air voids for the Center for Highway Research specimens of the Eagle Lake gravel mixtures is shown in Fig 7, which indicates an AVR design optimum asphalt content of about 4.5 percent.

The AVR design asphalt content obtained by the State Department of Highways and Public Transportation was approximately 4.7 percent. However, based on previous experience, the State Department of Highways and Public Transportation (DHT) increased the asphalt content for the plant mixture to 4.8 percent.

Lubbock Limestone Mixtures

The laboratory AVR relationships for the Center for Highway Research (CFHR) and the plant mixed specimens of the Lubbock limestone are shown in Fig 8. The AVR design optimum asphalt content was about 7.3 percent and the AVR design optimum for the plant mixture was about 6.8 percent. However,

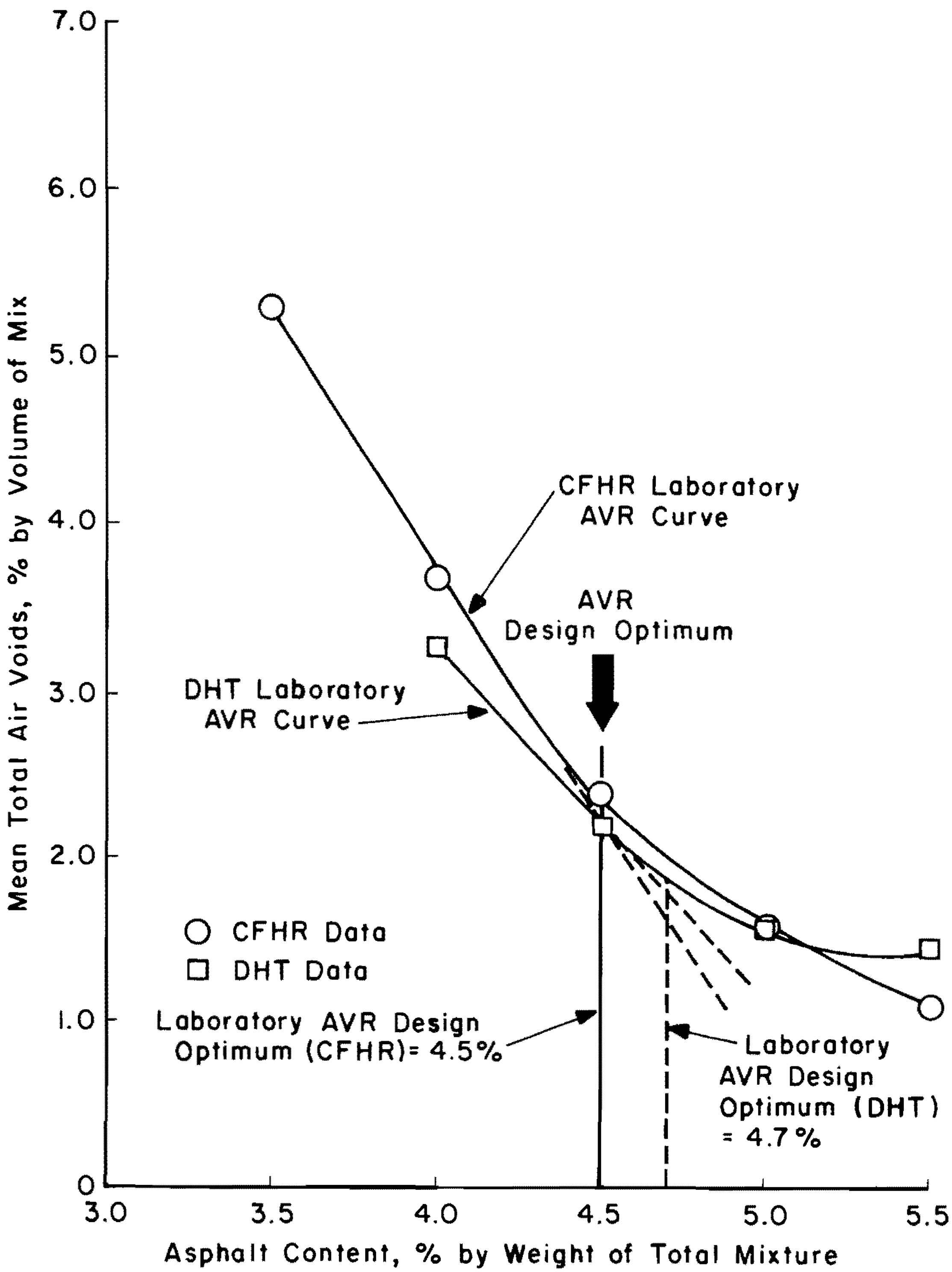


Fig 7. Relationship between asphalt content and total air voids for Eagle Lake gravel mixtures.

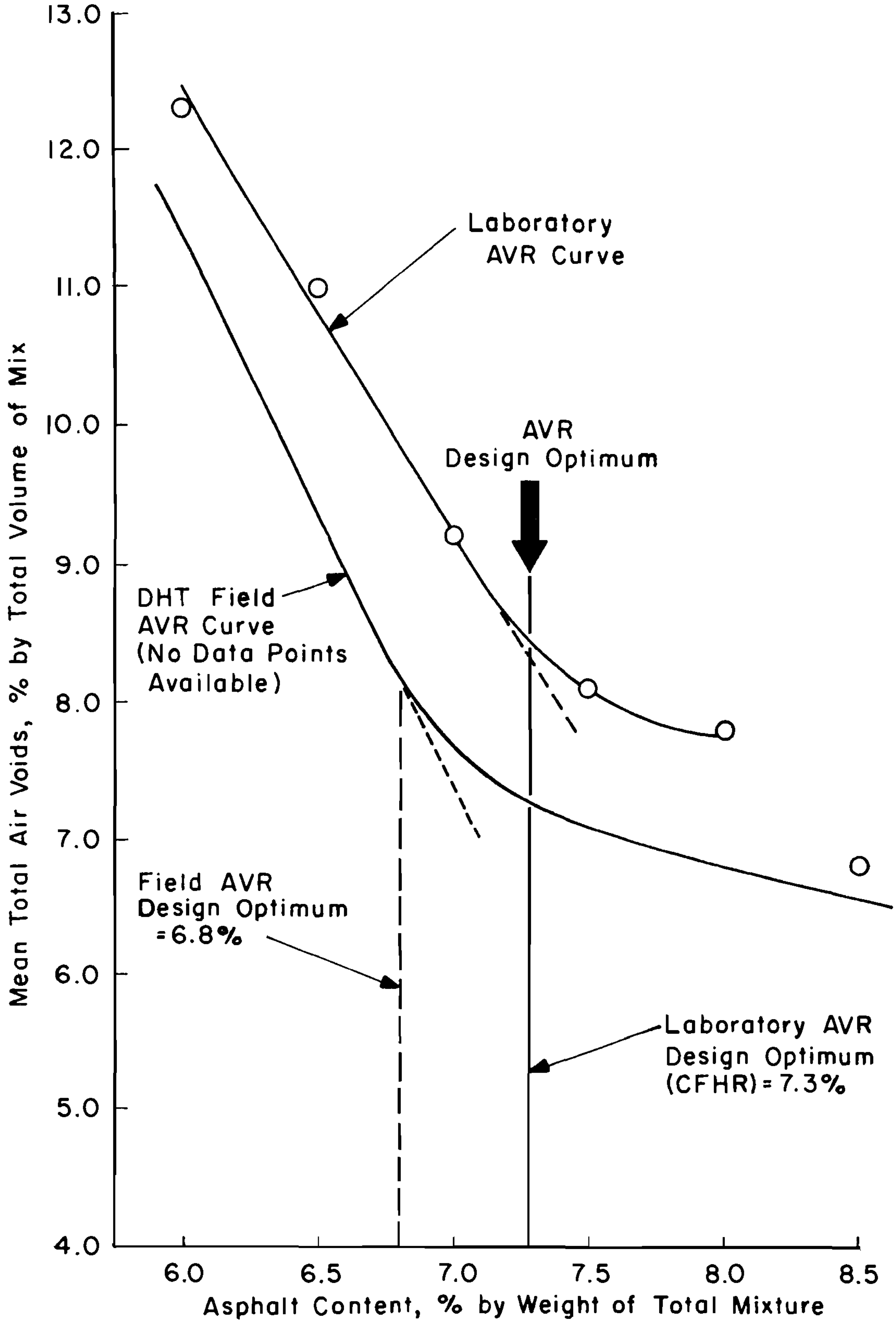


Fig 8. Relationship between asphalt content and total air voids for Lubbock limestone mixtures.

based on previous experience with the material, the DHT increased the asphalt content of the plant mixture to 7.3 percent.

Lufkin Sand Mixture

The relationship between asphalt content and air voids for the Lufkin sand mixture was different than for other mixtures since the relationship consisted of two straight lines (Fig 9). The design asphalt content of 7.5 percent, established by the intersection of these two lines, was used rather than a slightly higher value because, based on the appearance of the mixture, it was felt that a higher asphalt content would result in an overlubricated mixture.

DENSITY

The relationships between density and asphalt content for the three mixtures are shown in Figs 10, 11, and 12. The AVR densities were generally greater than the densities obtained for specimens cut from the top and bottom of the compacted specimen. This can be explained by the fact that the AVR densities were determined while the specimens were still in the mold and subjected to a compressive stress of 3450 kPa (500 psi) while the densities for the top and bottom specimens were determined after the large compacted specimens had been removed from the mold and sawed, which allowed some expansion of the specimen.

The maximum in-mold AVR densities for the three materials were 2395 kg/m^3 (149.5 pcf) for the Eagle Lake gravel mixture, 2197 kg/m^3 (137.2 pcf) for the Lubbock limestone mixture, and 2326 kg/m^3 (145.2 pcf) for the Lufkin sand mixture. Similarly, the average of the top and bottom densities ranged from 2190 kg/m^3 (136.7 pcf) for the Lubbock limestone mixture to 2369 kg/m^3 (147.9 pcf) for the Eagle Lake gravel mixture. The in-mold AVR densities for individual specimens are listed in Appendix D.

The optimum asphalt contents for maximum in-mold AVR density were 4.9, 7.6, and 7.5 percent for the Eagle Lake, Lubbock, and Lufkin mixtures, respectively (Figs 10, 11, and 12). As previously noted, these asphalt contents are greater than or equal to the laboratory AVR optimums (Ref 1). However, the optimum asphalt contents for maximum density for the top and bottom specimens were less than the optimum asphalt contents for maximum in-mold AVR density,

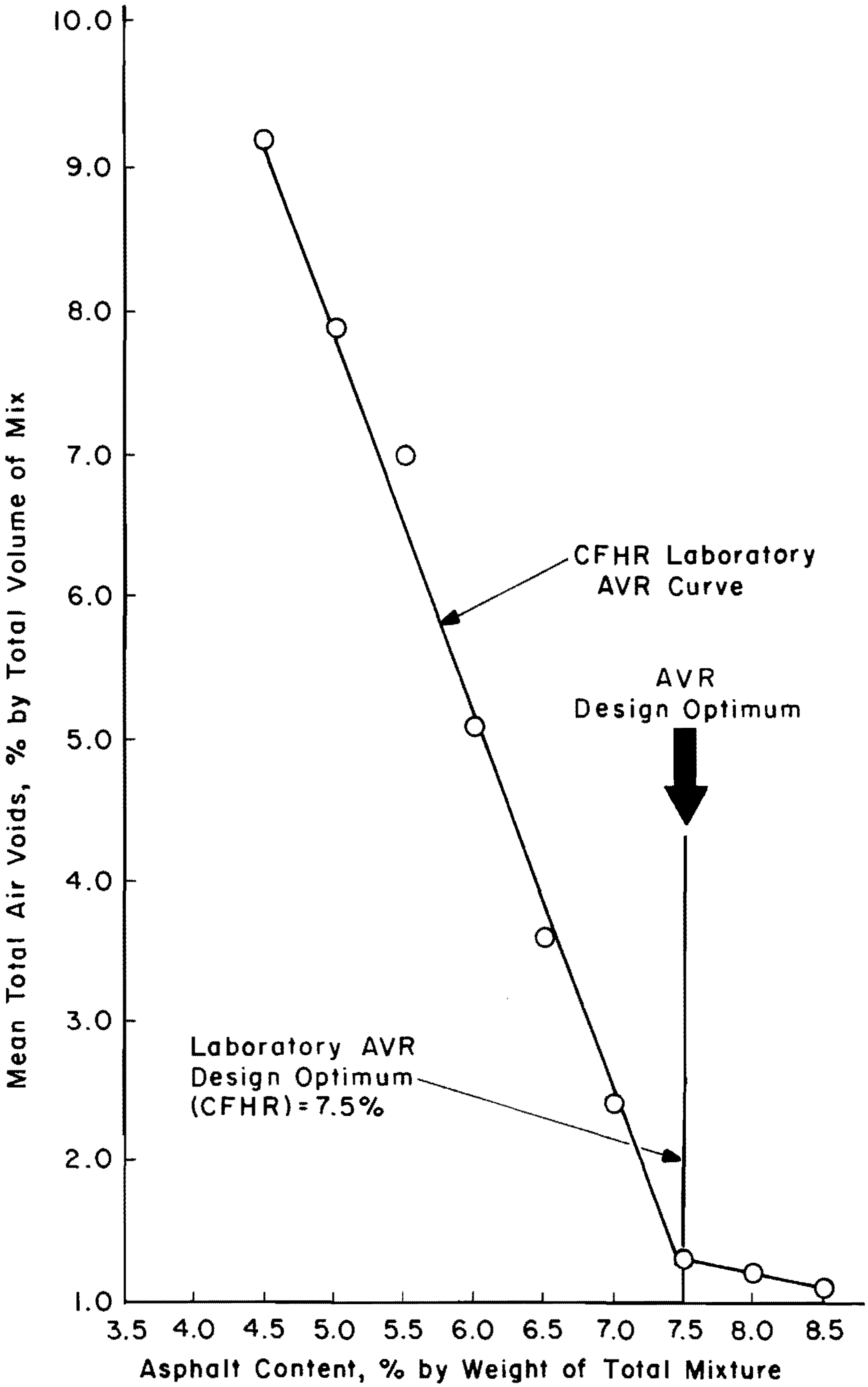


Fig 9. Relationship between asphalt content and total air voids for the Lufkin sand mixtures.

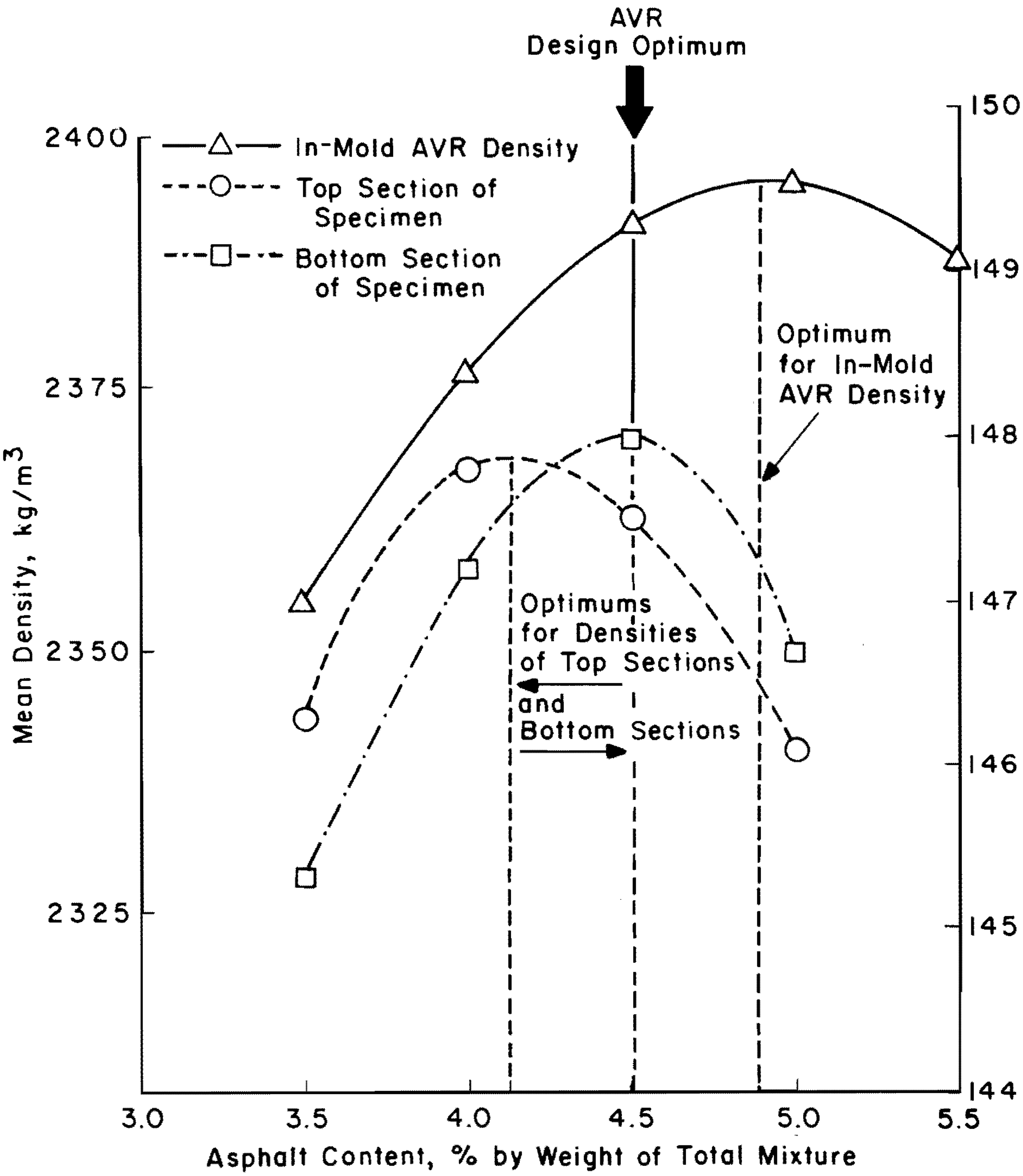


Fig 10. Relationships between asphalt content and density for Eagle Lake gravel mixtures.

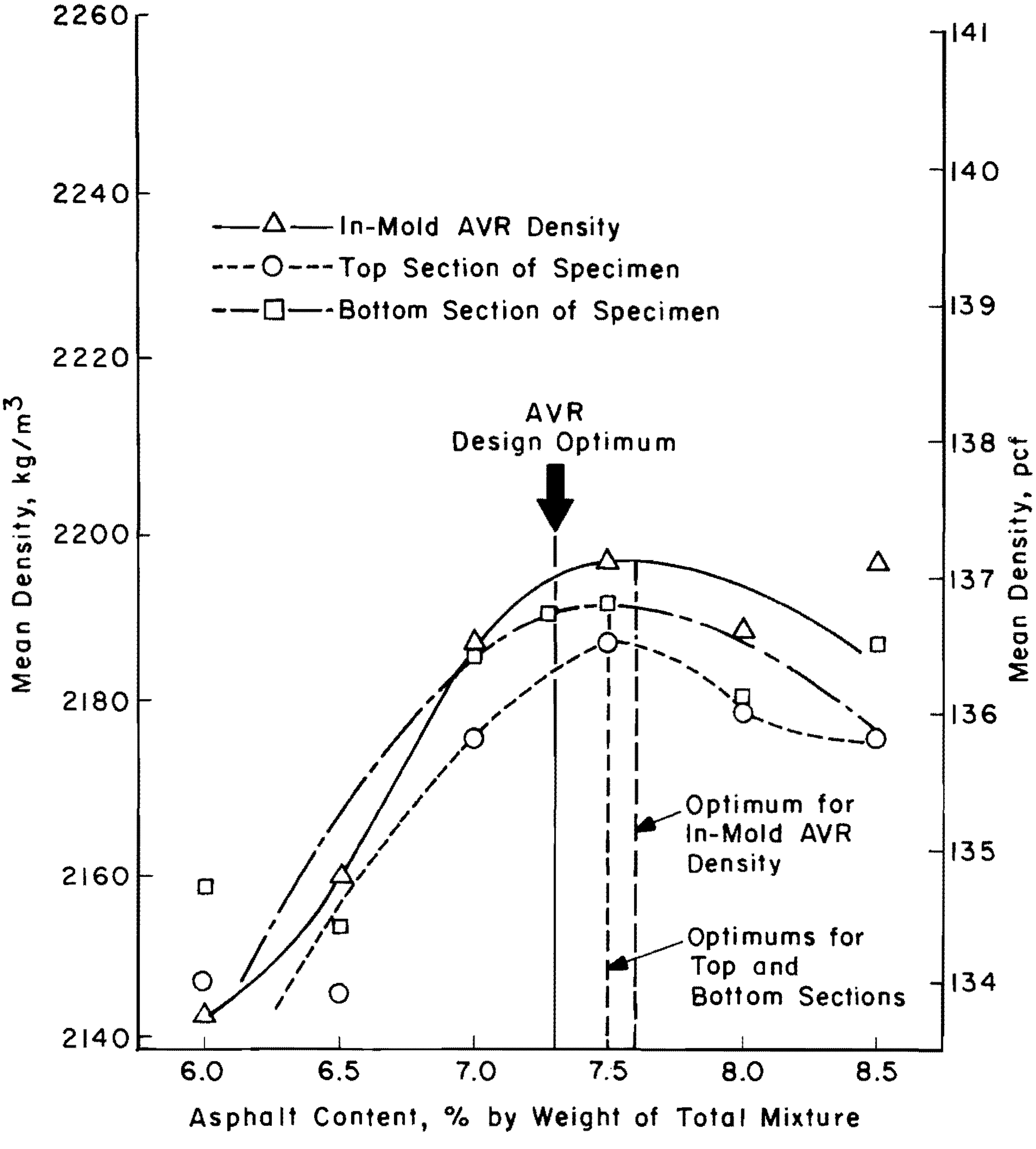


Fig 11. Relationships between asphalt content and density for Lubbock limestone mixtures.

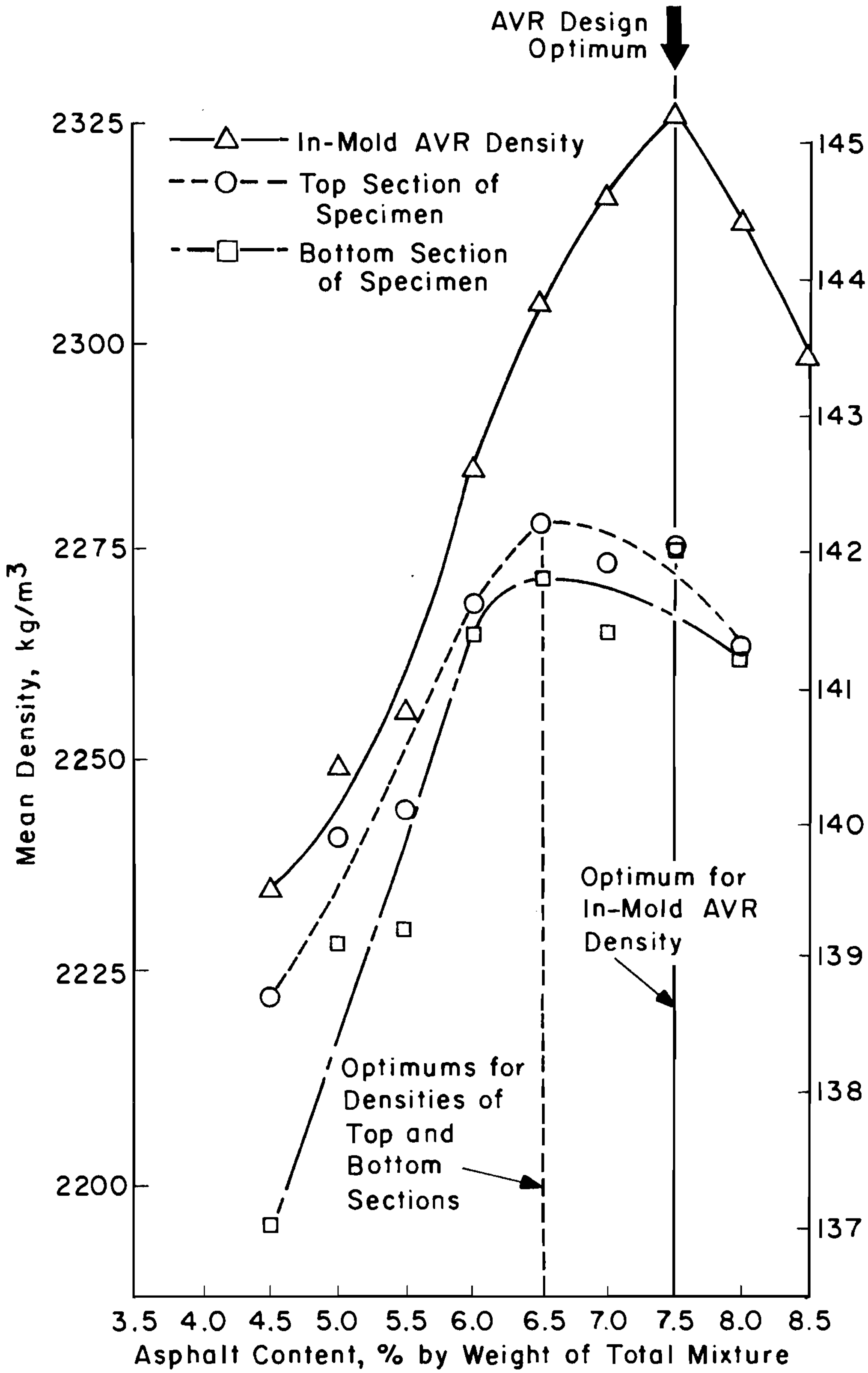


Fig 12. Relationships between asphalt content and density for Lufkin sand mixtures.

indicating that the optimum for maximum density was dependent on the type of density and position within the specimen.

UNCONFINED COMPRESSION TESTS

Unconfined compression tests were performed on specimens at or near the AVR optimum asphalt content for both the fast and the slow rates of deformation in order to determine whether the mixture satisfied the unconfined compressive strength requirements of Test Method Tex-126-E (Table 2). The unconfined compressive strengths for all three mixtures (Figs 13, 14, and 15) did not satisfy strength specifications.

The unconfined compressive strengths for the Eagle Lake gravel mixture were far below the minimum strength requirements for the poorest grade of blackbase (Grade 3) at both the fast and the slow speeds (Fig 13).

The Lubbock limestone mixture exceeded the strength requirements at the slow speed but failed to meet the strength requirements at the fast speed in the CFHR tests (Fig 14). However, the unconfined compression tests performed by the State Department of Highways and Public Transportation on the Lubbock limestone indicated that the laboratory mixture satisfied the specified strength requirements for both speeds. It should be noted that the gradations of the laboratory and plant mixtures for the Lubbock limestone were significantly different. Since this investigation involved mixtures with the gradation of the plant mixture, differences in unconfined compressive strengths were to be expected.

Finally, the Lufkin sand mixture failed to satisfy the minimum strength requirements for the poorest grade of blackbase at the fast loading rate. It can be seen from Fig 15 that the unconfined compressive strength at the fast loading rate was much less than the required strength. In addition, at the design asphalt content the slow-speed strengths failed to meet minimum strength requirements; however, the strengths were satisfactory for a small range of asphalt contents that were less than the AVR design optimum asphalt content, indicating that a more satisfactory mixture might result at lower asphalt contents. It should be noted that the pressure pycnometer, which was used to saturate the specimens, produced severe damage to the specimens containing Lufkin sand.

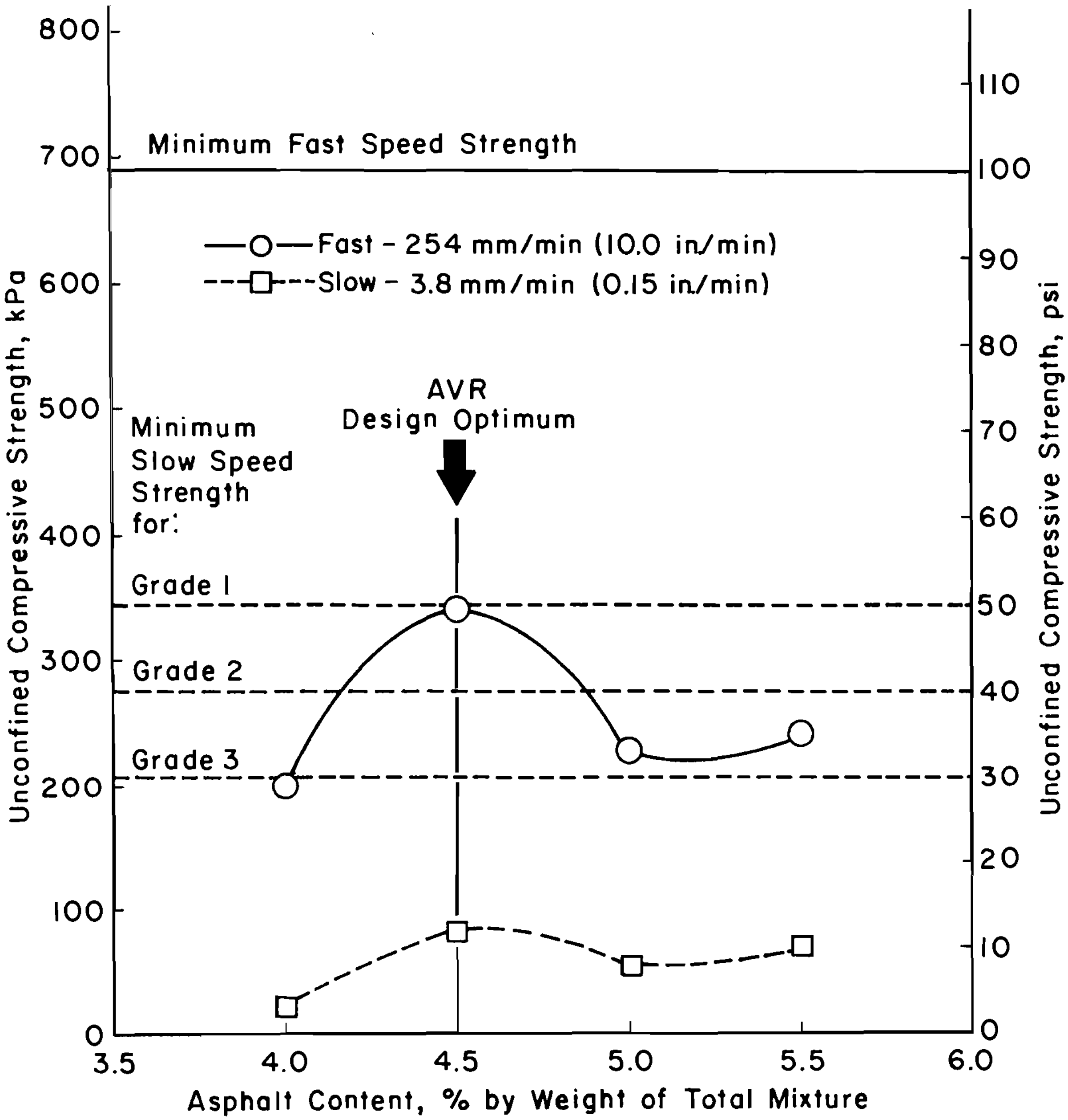


Fig 13. Effect of asphalt content and rate of deformation on unconfined compressive strength for Eagle Lake gravel mixtures.

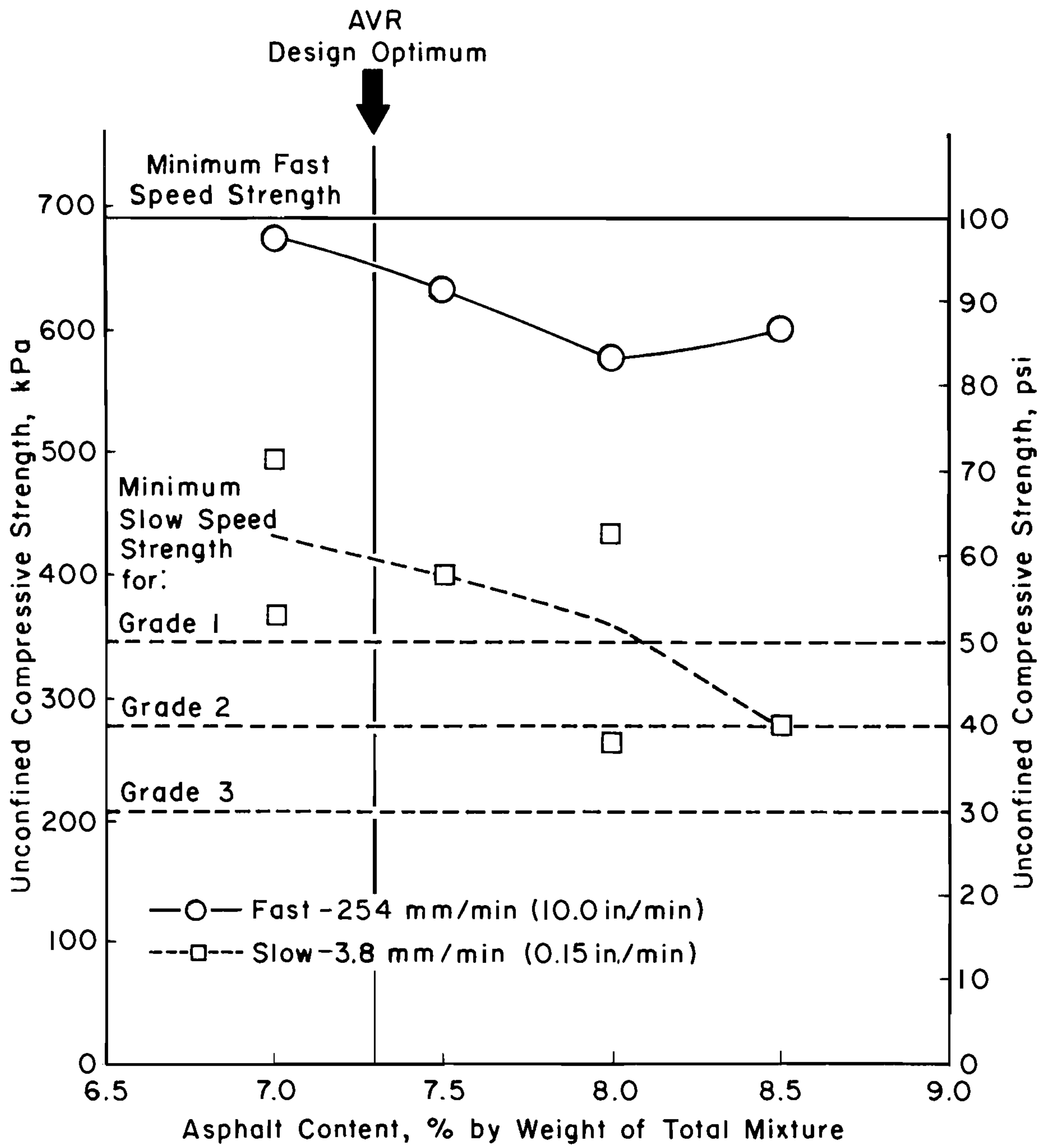


Fig 14. Effect of asphalt content and rate of deformation on unconfined compressive strength for Lubbock limestone mixtures.

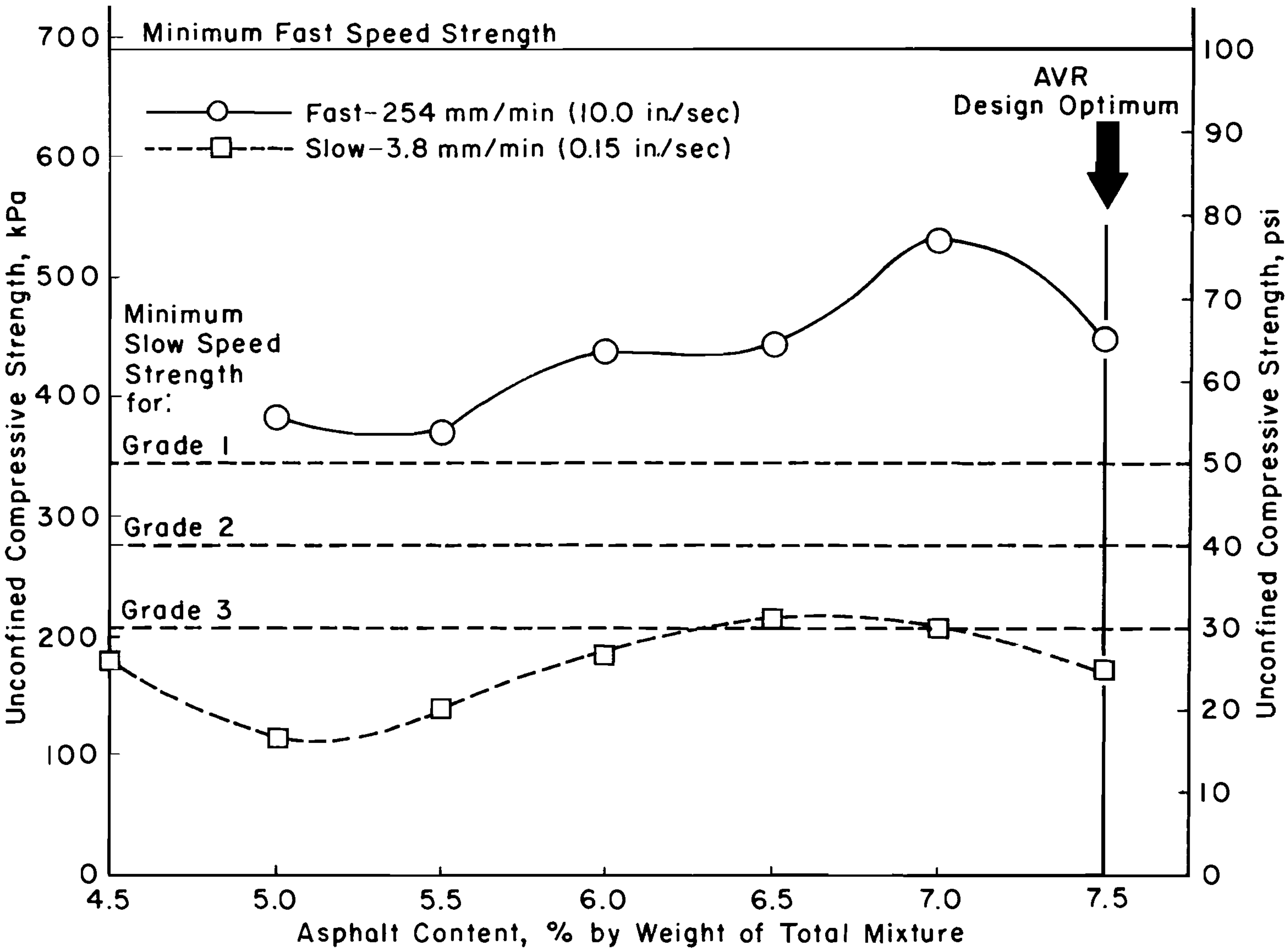


Fig 15. Effect of asphalt content and rate of deformation on unconfined compressive strength for Lufkin sand mixtures.

In conclusion, according to Test Method Tex-126-E all mixtures failed to satisfy minimum unconfined compressive strength standards. Nevertheless, according to district personnel of the Department of Highways and Transportation all mixtures have provided satisfactory pavement performance.

STATIC INDIRECT TENSILE TEST RESULTS

Two engineering properties, tensile strength and static modulus of elasticity, were estimated using the static indirect tensile test. The ultimate tensile strength and static modulus of elasticity for individual specimens are presented in Appendix E, along with calculated values of Poisson's ratio.

Tensile Strength

For the range of temperatures studied, the optimum asphalt content for ultimate tensile strength was found to increase slightly with a decrease in temperature for all three mixtures (Figs 16, 17, and 18), which agrees with previous findings (Refs 3, 13, 14, and 15).

The optimum asphalt contents for the Eagle Lake gravel mixture ranged from 4.0 percent at 38°C (100°F) to about 4.2 percent at 10°C (50°F) (Fig 16). For the same temperature range the optimum asphalt contents for the Lubbock limestone ranged from about 6.1 to 7.1 percent (Fig 17). The Lufkin sand has optimum asphalt contents ranging from 5.0 percent at 38°C (100°F) to 6.0 percent at 24°C (75°F) (Fig 18). However, even though optimums were found at 24 and 38°C (75 and 100°F) in the Lufkin sand mixture, the ultimate tensile strengths were essentially independent of asphalt content at these temperatures.

The optimum asphalt contents for ultimate tensile strength for all mixtures and temperatures were less than the optimum AVR design asphalt content by as much as 0.2 to 2.5 percentage points, depending on the material and temperature. For the Eagle Lake and Lubbock mixtures, the optimum asphalt contents for tensile strength were from about 0.2 to 1.1 percentage points less than the AVR design optimum. For the Lufkin sand mixture alone the optimums ranged from 1.5 to 2.5 percentage points less than the AVR design optimum asphalt content.

The maximum tensile strength for the Eagle Lake gravel mixture ranged from about zero to 25 percent greater than the tensile strength at the laboratory AVR optimum; for the Lubbock limestone mixture the maximum tensile

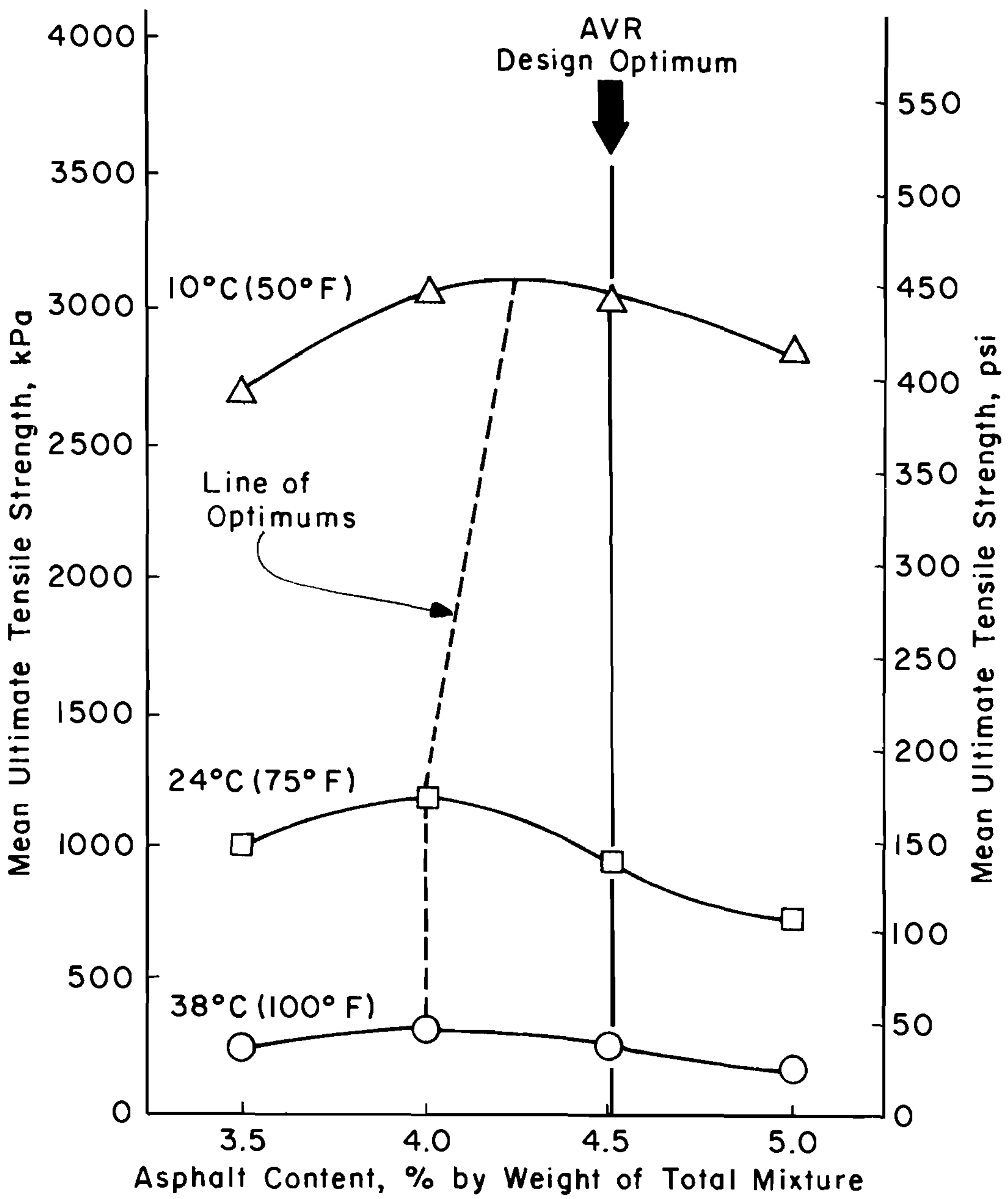


Fig 16. Effect of asphalt content and temperature on ultimate tensile strength for Eagle Lake gravel mixtures.

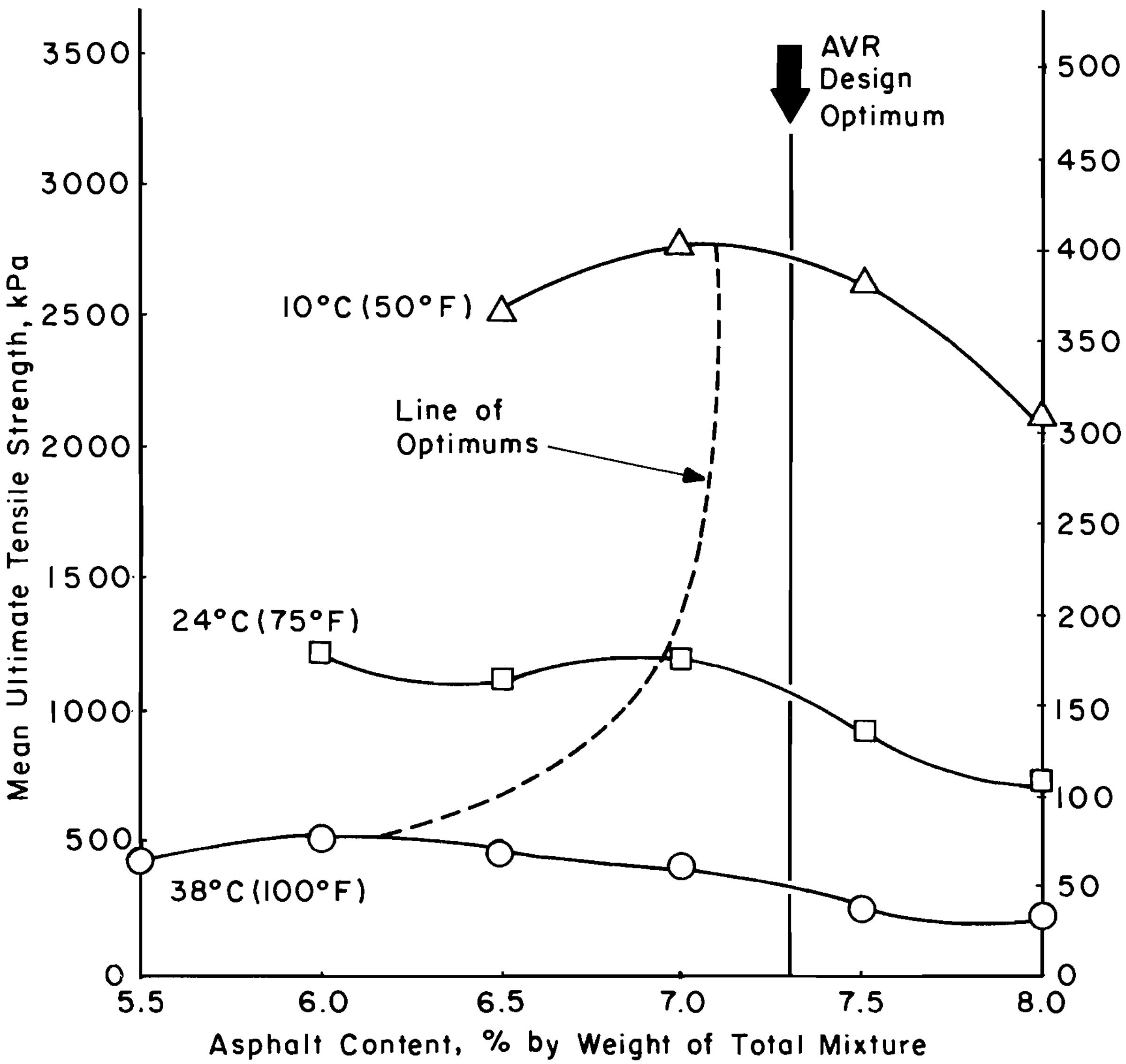


Fig 17. Effect of asphalt content and temperature on ultimate tensile strength of Lubbock limestone mixtures.

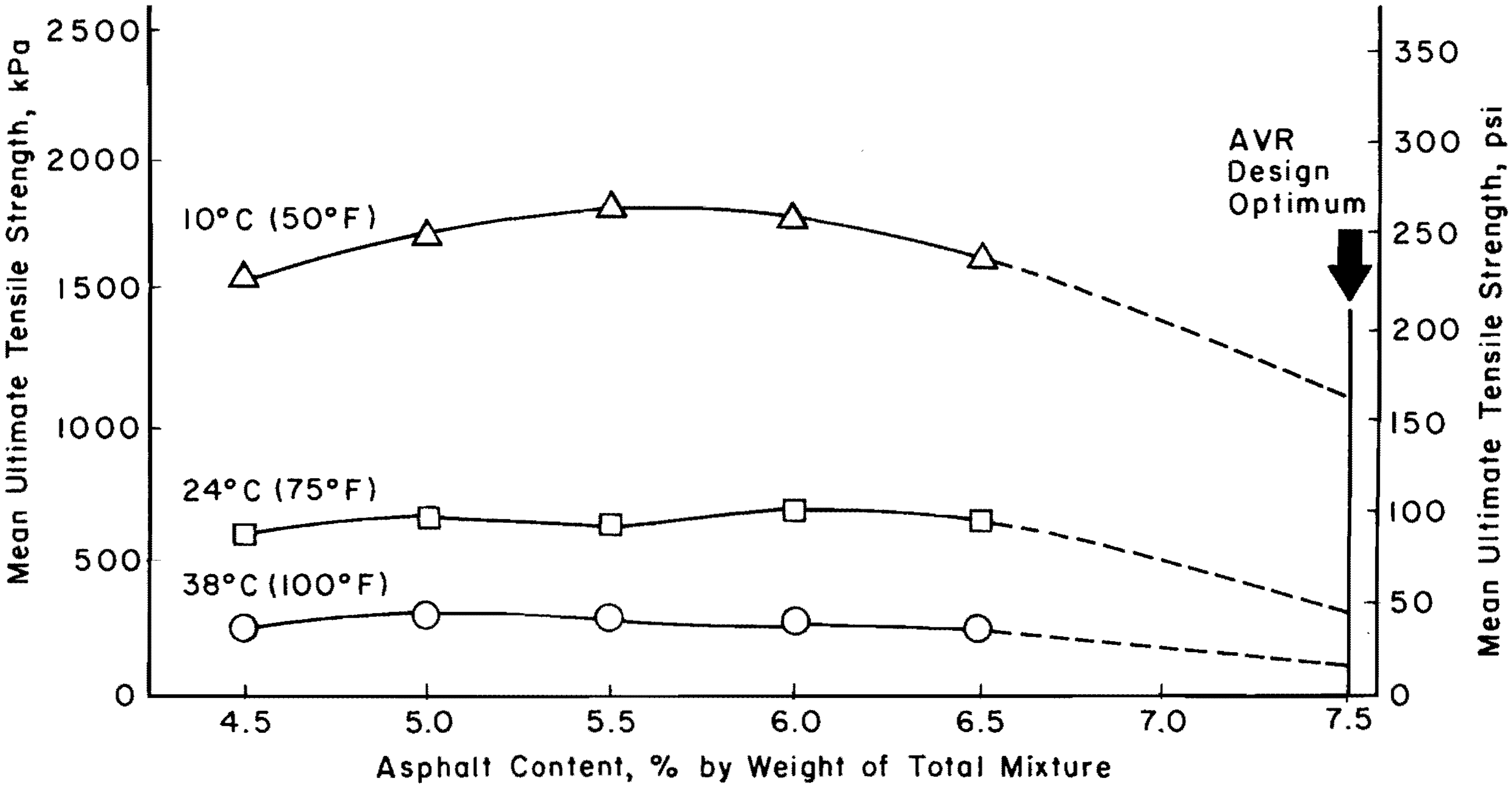


Fig 18. Effect of asphalt content and temperature on ultimate tensile strength for Lufkin sand.

strength ranged from about zero to 50 percent greater than the value at the AVR design optimum; and for the Lufkin sand mixture, depending on the temperature, the estimated maximum tensile strength was from 100 to 200 percent greater than the estimated values at the AVR design optimum of 7.5 percent. From the relationships between asphalt content and ultimate tensile strength it was found that at higher temperatures the effects of asphalt content were small, i.e., the tensile strengths were essentially independent of asphalt content, while at low temperatures the asphalt content had a significant effect on tensile strength. There was also a general trend found in each of the materials, indicating that the optimum asphalt content for maximum tensile strength increased with decreasing temperature for the range of testing temperatures. In addition, the relationships were essentially symmetrical, indicating that the reduction in tensile strength wet of optimum was the same as that dry of optimum.

For comparison purposes, the ultimate tensile strength of each material for each temperature is shown in Fig 19. At 10°C (50°F) the Eagle Lake gravel mixture had the highest tensile strength, with a value of 3100 kPa (450 psi). The Lubbock limestone mixture was slightly weaker, 2750 kPa (400 psi), while the strength of the Lufkin sand mixture was significantly less, 1850 kPa (270 psi). At 38°C (100°F) the Lubbock limestone mixture had the greatest tensile strength, with a value of about 520 kPa (75 psi), while the Eagle Lake gravel and Lufkin sand mixtures had the same tensile strengths, about 300 kPa (45 psi). These relationships are typical of those found in previous studies.

Static Modulus of Elasticity

Maximum static moduli of elasticity for the three materials at 10°C (50°F) ranged from 1650×10^3 kPa (239×10^3 psi) for the Lufkin sand mixture to 4700×10^3 kPa (681×10^3 psi) for the Eagle Lake gravel mixture. At 38°C (100°F) the values ranged from about 100×10^3 kPa (11.4×10^3 psi) for the Lufkin sand mixture to about 700×10^3 kPa (101×10^3 psi) for the Lubbock limestone mixture.

Optimum asphalt contents for maximum static modulus of elasticity existed for all mixtures and temperatures (Figs 20, 21, and 22). At 24 and 38°C (75 and 100°F), however, the optimums for the Lufkin sand mixture were poorly defined. The optimum asphalt contents for the Eagle Lake gravel mixture ranged from 3.9 percent at 24°C (75°F) to 4.4 percent at 10°C (50°F) (Fig 20).

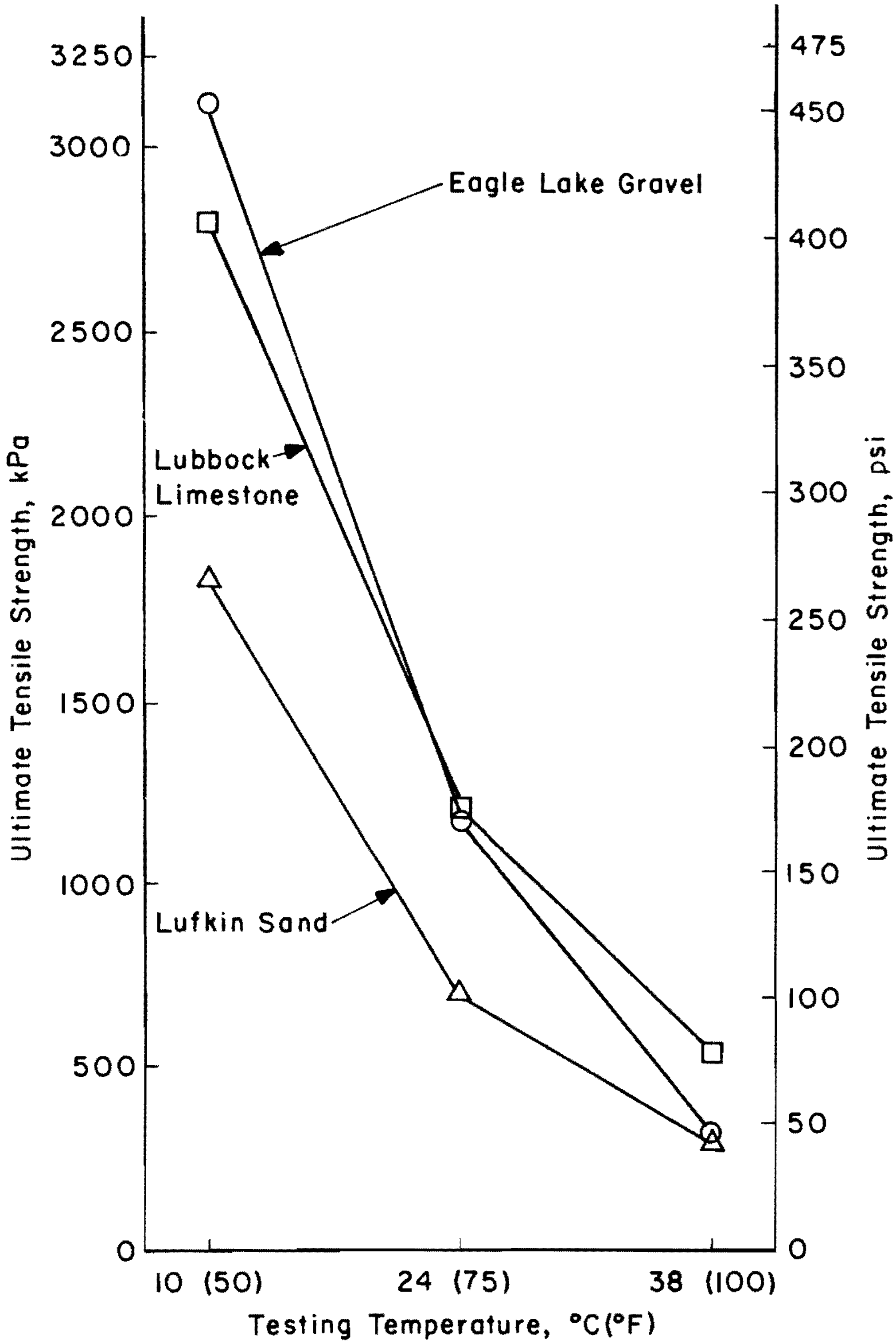


Fig 19. Relationship between testing temperature and ultimate tensile strength for mixtures at optimum asphalt content for maximum tensile strength.

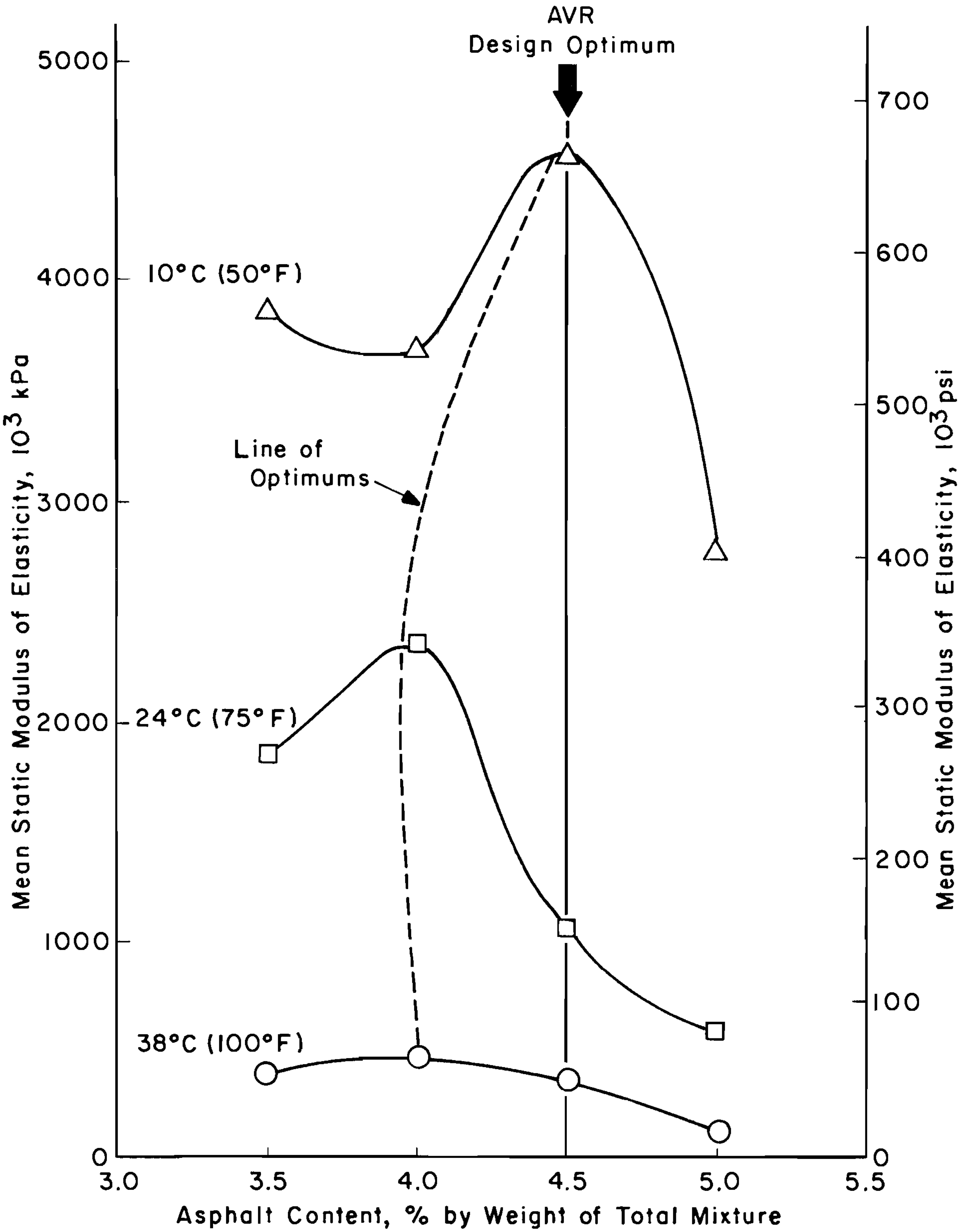


Fig 20. Effect of asphalt content and temperature on static modulus of elasticity for Eagle Lake gravel mixtures.

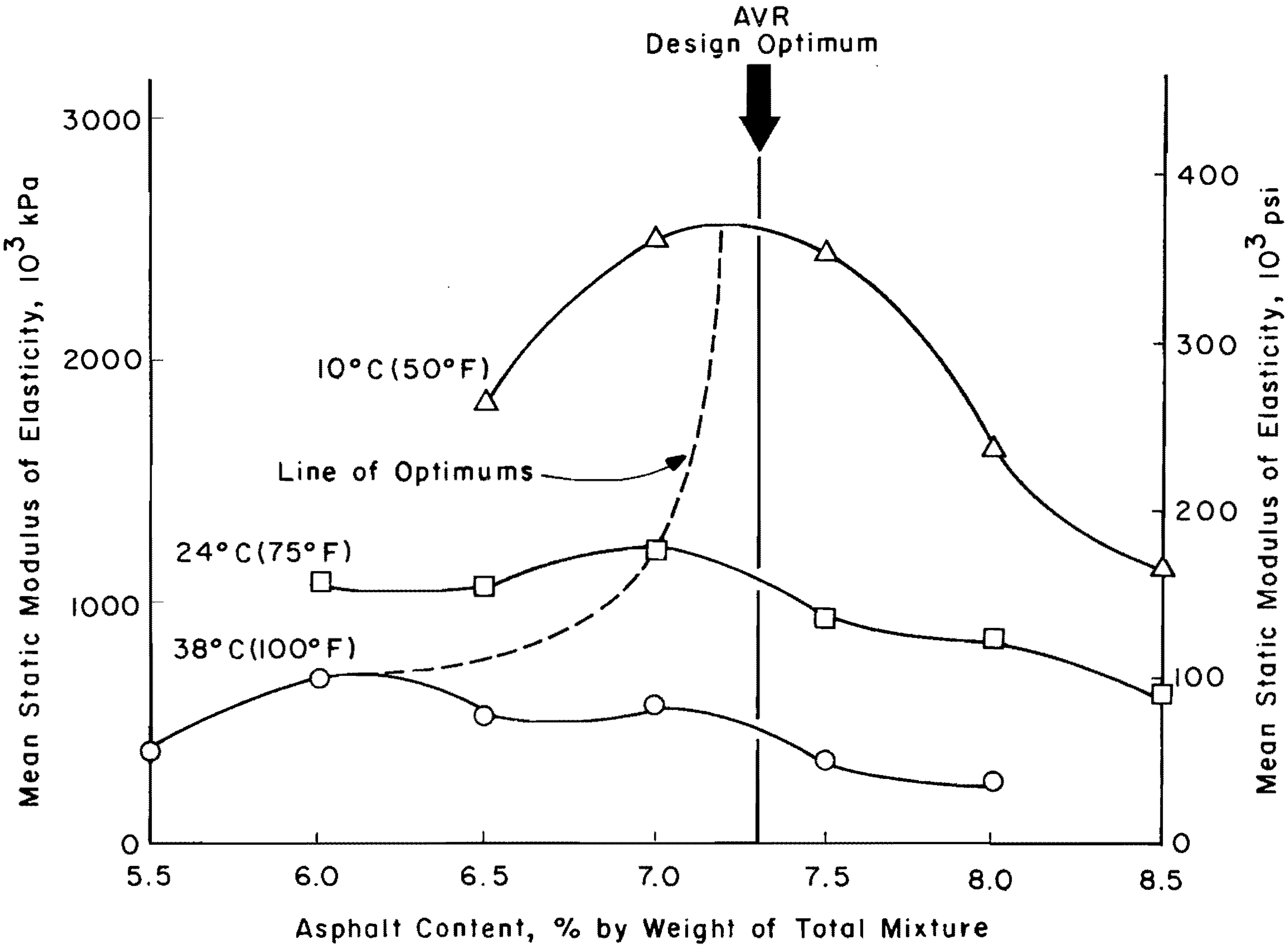


Fig 21. Effect of asphalt content and temperature on static modulus of elasticity for Lubbock limestone mixtures.

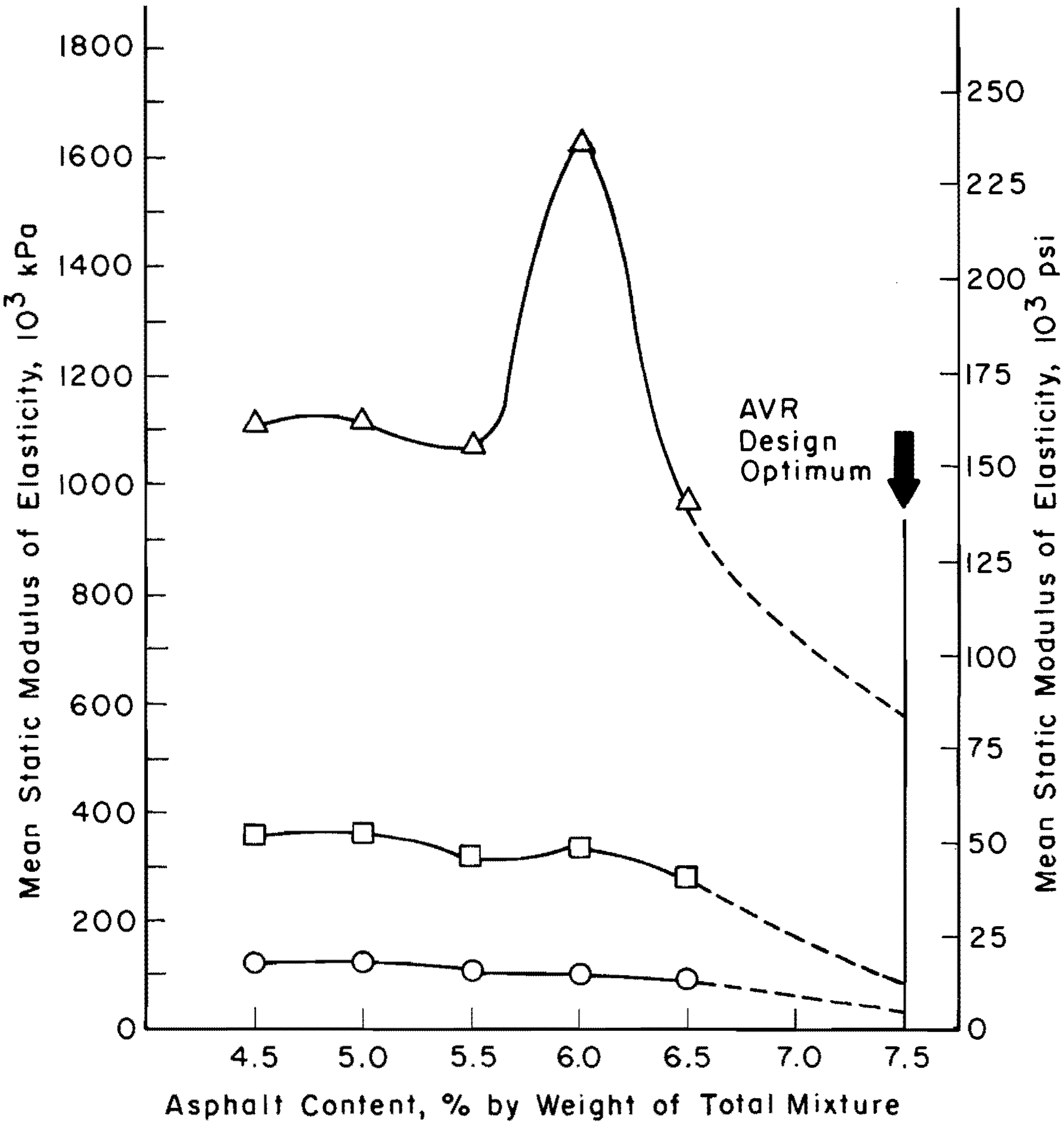


Fig 22. Effect of asphalt content and temperature on static modulus of elasticity for Lufkin sand mixtures.

For the Lubbock limestone mixture the optimums ranged from 6.2 percent at 38°C (100°F) to 7.2 percent at 10°C (50°F) (Fig 21). The optimum asphalt contents for the Lufkin sand mixture ranged from a poorly defined value of about 4.8 percent at 24°C (75°F) to 6.0 percent at 10°C (50°F) (Fig 22).

The optimum asphalt contents for maximum static moduli of elasticity for all mixtures and temperatures were less than the AVR design optimum asphalt contents by as much as 0.1 to 2.7 percentage points, depending on the mixture and temperature. The optimums for static moduli of elasticity for the Eagle Lake and Lubbock mixtures were from 0.1 to 1.1 percentage points less than the laboratory AVR design optimum. For the Lufkin sand mixture alone the optimums were from 1.5 to 2.7 percentage points less than the AVR design optimum.

As a result of these differences, the maximum static modulus of elasticity for the Eagle Lake gravel mixture ranged from about 15 to 125 percent greater than the value at the AVR optimum. For the Lubbock limestone mixture, the value of maximum static modulus of elasticity did not exceed the value at the AVR optimum by more than about 25 percent. Although the static modulus of elasticity was not obtained at the AVR optimum for the Lufkin sand mixture, it can be seen from Fig 22 that the maximum values of static modulus of elasticity are probably significantly greater than the values of the AVR optimum, depending on the temperature.

In addition, the effect of asphalt content decreased as temperature increased, i.e., asphalt content did not have a significant effect on the static modulus of elasticity at high temperatures. Also, the optimum asphalt content for static modulus of elasticity generally increased with decreasing temperature. Finally, it was found that the change in the static modulus of elasticity on the wet and dry sides of the optimum asphalt content was dependent on material and temperature and no consistent trends were observed.

REPEATED-LOAD INDIRECT TENSILE TEST RESULTS

Repeated-load indirect tensile tests were conducted to evaluate the fatigue life, resilient modulus of elasticity, and resistance to permanent deformation of the materials being studied. Physical properties of the specimens and repeated-load test results for individual specimens are listed in Appendix F.

Fatigue Life

The relationship between stress difference and fatigue life for each material at 24°C (75°F) is shown in Fig 23. Stress difference was assumed to be equal to approximately four times the applied tensile stress (Ref 16). The two data points for each material indicated in Fig 23 represent the average values of maximum fatigue life at the respective stress actually applied to the specimen. Since the relationship has been shown to be linear in previous studies (Refs 3, 4, 5, 6, and 7), it was assumed to be linear for these materials.

From Fig 23 it can be seen that the Lubbock limestone mixture has the greatest resistance to fatigue, followed by the Lufkin sand and the Eagle Lake gravel mixtures. Also shown as having similar relationships are a gravel and limestone of the same gradation which were reported by Adedimila and Kennedy (Ref 3). Relationships between fatigue life and asphalt content were developed for each material to study the effects of temperature and asphalt content. Using Fig 23, the applied stress, or stress difference, was normalized to eliminate the effect of differences in applied stress for each material. The estimated fatigue life for any set of conditions was determined at a stress difference of 400 kPa (58 psi) or a tensile stress of 100 kPa (14.5 psi) by assuming a linear relationship between stress difference and fatigue life. This value was chosen to evaluate the effect of temperature and asphalt content, because it minimized the need to extrapolate the fatigue life-stress relationships. The resulting relationships are shown in Figs 24, 25, and 26.

Correlation analyses, also, were made to determine the relationships between actual fatigue life and both the initial tensile strain and the applied stress-strength ratio. These relationships have been used by other investigators (Refs 3, 4, 5, 10, and 14) to estimate the fatigue life of asphalt mixtures and would have decreased the amount of testing required to obtain fatigue life estimates. The resulting correlations, however, were found to have very low coefficients of determination and, therefore, were of questionable value.

An optimum asphalt content for maximum fatigue life was found for all three mixtures and all test conditions studied (Figs 24, 25, and 26), which is consistent with previous findings (Refs 3, 4, 8, 9, 10, and 11).

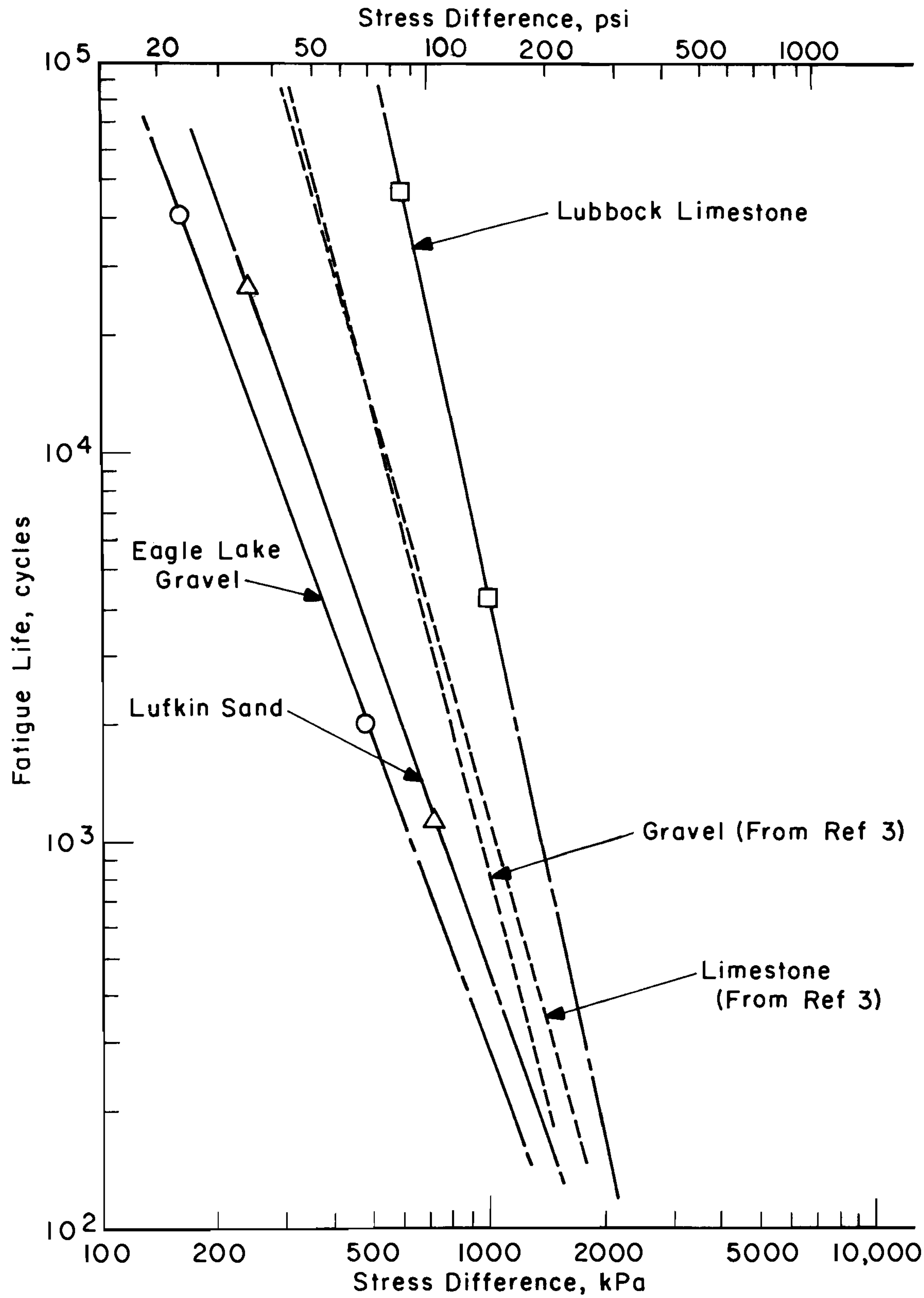


Fig 23. Effect of stress difference on maximum fatigue life at 24°C (75°F).

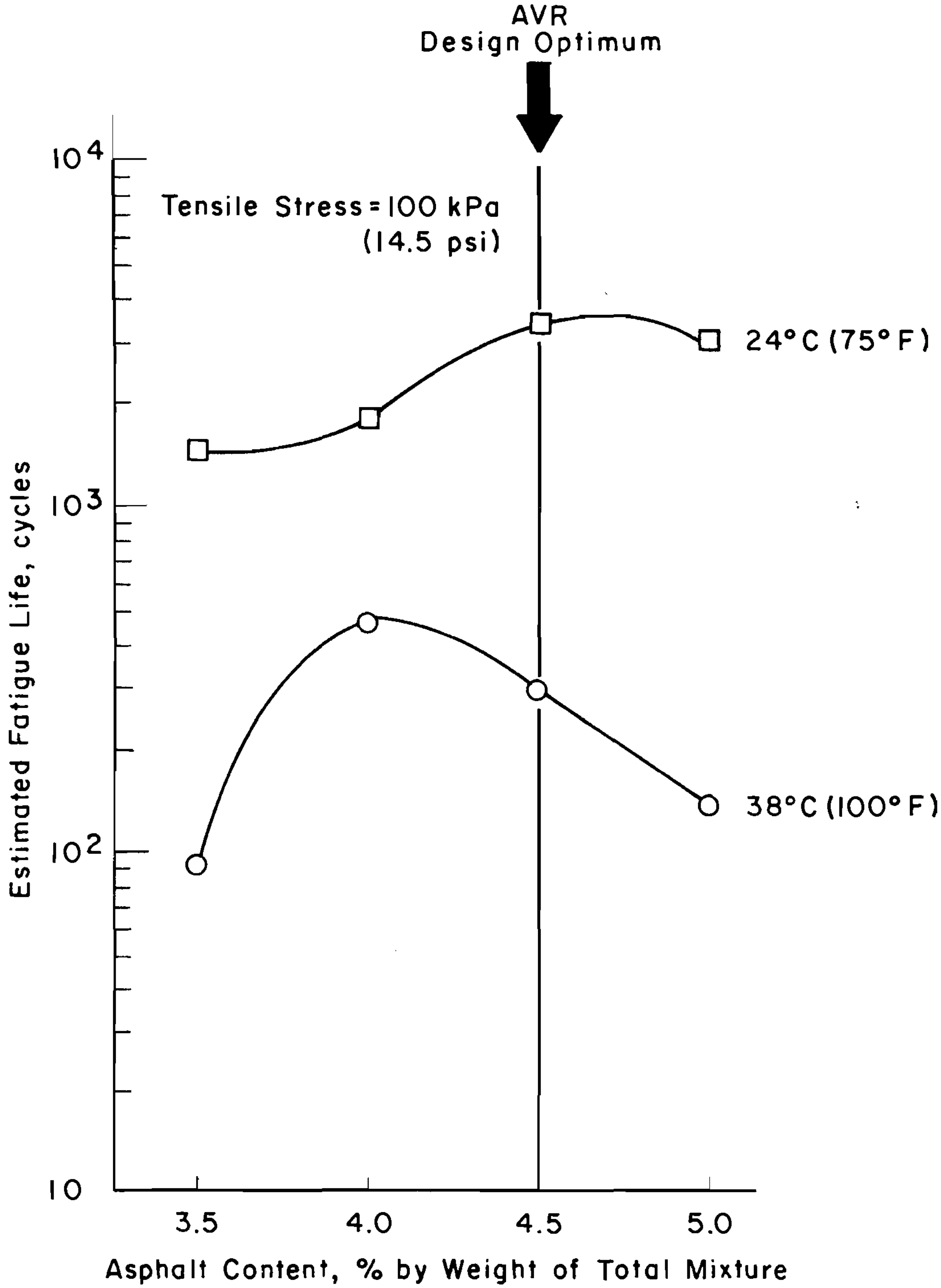


Fig 24. Relationships between asphalt content and fatigue life at 100 kPa (14.5 psi) for Eagle Lake gravel mixtures.

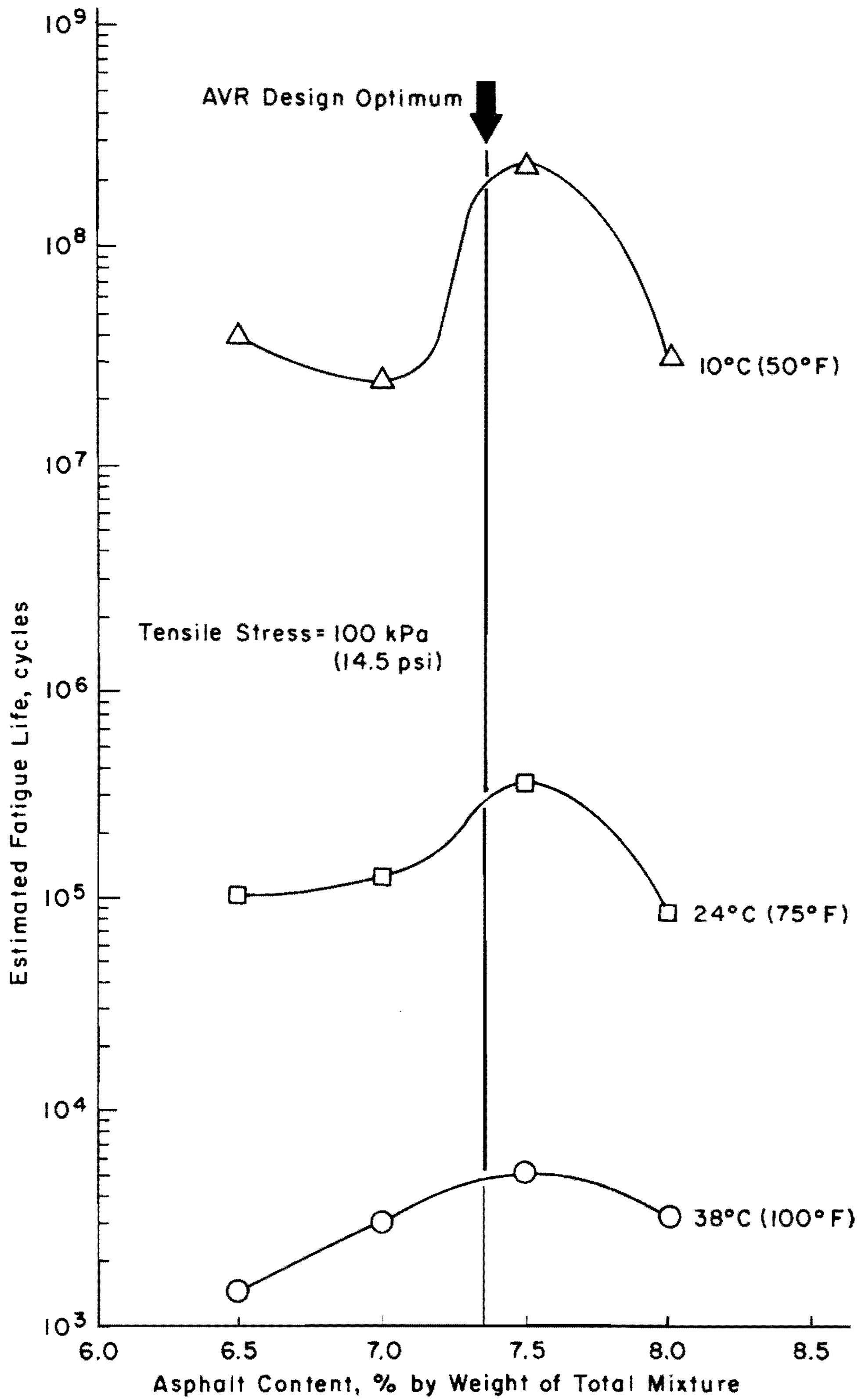


Fig 25. Relationships between asphalt content and fatigue life at 100 kPa (14.5 psi) for Lubbock limestone mixtures.

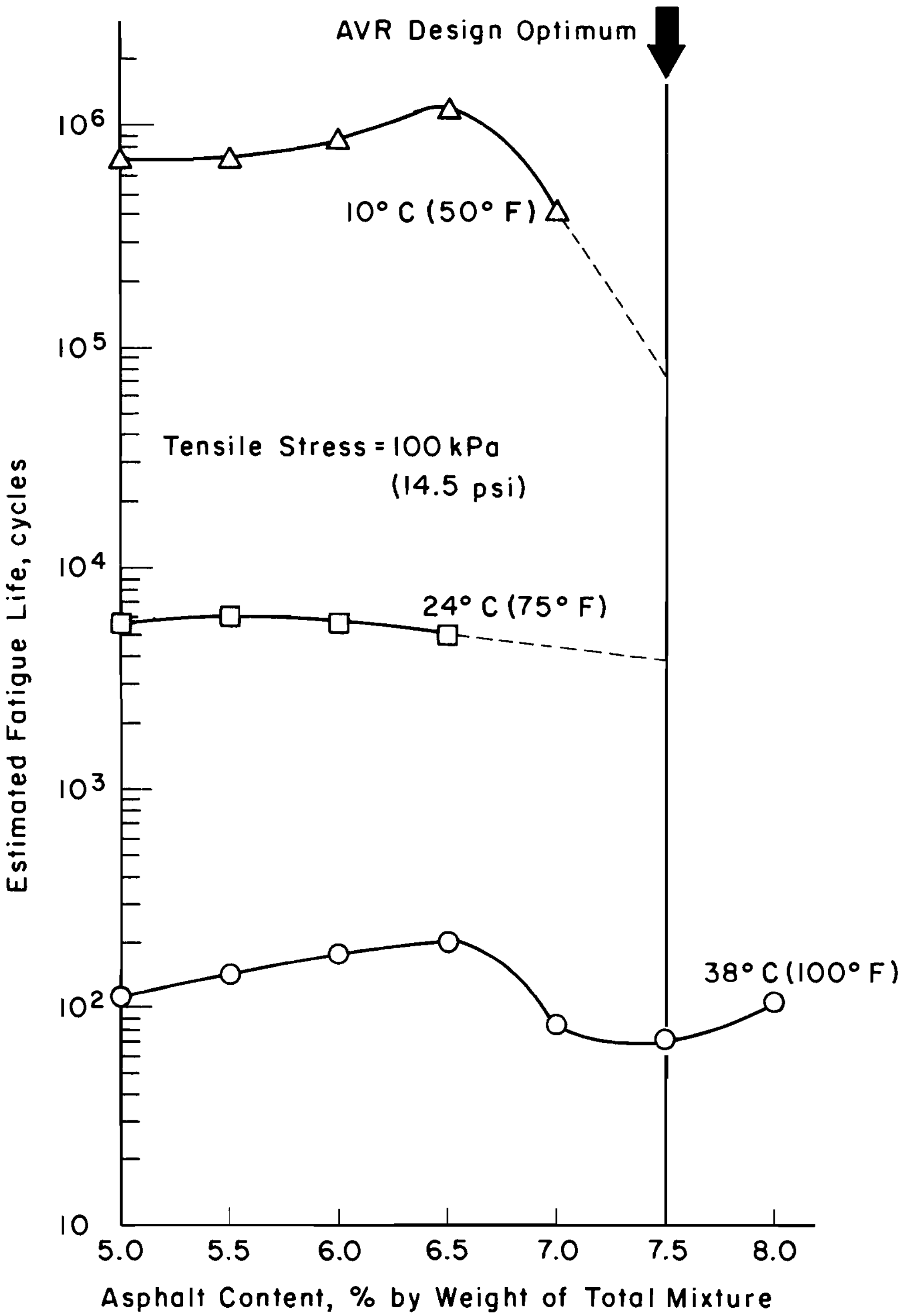


Fig 26. Relationships between asphalt content and fatigue life at 100 kPa (14.5 psi) for Lufkin sand mixtures.

Depending on the temperature, the optimum asphalt content for maximum fatigue life of the Eagle Lake gravel ranged from 4.0 to 4.6 percent (Fig 24), which is from 0.1 percentage point more to 0.5 percentage point less than the AVR design optimum. For the Lubbock limestone mixture (Fig 25) the optimum asphalt content was 7.5 percent, regardless of temperature, which was approximately 0.2 percentage point greater than the AVR design optimum. However, for the Lufkin sand mixture (Fig 26) the optimum ranged from about 5.5 to 6.5 percent, which is from 1.0 to 2.0 percentage points less than the AVR design optimum. Thus, the optimum asphalt content for maximum fatigue life tended to be less than the AVR design optimum asphalt content for the Lufkin sand mixture.

Because of these differences, the maximum fatigue life for the Eagle Lake gravel mixture was as much as 60 percent greater than the fatigue life at the AVR optimum, depending on the temperature. For the Lubbock limestone mixture, the values of maximum fatigue life were 15 to 200 percent greater than the fatigue life at the AVR design optimum, with the larger differences occurring at the lower temperatures. Also, the percent difference between maximum fatigue life and the value at the AVR optimum for the Lubbock limestone mixture increased with decreasing temperature. For the Lufkin sand mixture, fatigue lives were not available at the AVR optimum except at 38°C (100°F). For this condition, the maximum fatigue life was about 150 percent greater than the fatigue at the AVR optimum. However, by estimating maximum fatigue life at 24°C (75°F) and 10°C (50°F), it can be seen (Fig 26) that the maximum fatigue life could be anywhere from 150 to 1000 percent greater than the value at the AVR design optimum.

It can also be noted that the optimum asphalt content for fatigue life is better defined at low temperatures; at the higher temperatures, the effect of asphalt content was not as significant. From Figs 24, 25, and 26 it can be seen that there was possibly a slight tendency for the optimum asphalt content for maximum fatigue life to increase with a decrease in temperature. In addition, the effect of asphalt content above and below the optimum value was essentially the same.

Resilient Modulus of Elasticity

While an optimum asphalt content for maximum resilient modulus of elasticity was evident for most of the mixtures studied, the actual value was not well defined, indicating that with this range asphalt content did not have a significant effect on resilient modulus. This agrees with the findings of other investigators (Refs 8 and 10).

The optimum asphalt contents for the Eagle Lake gravel mixture occurred only at 10 and 38°C (50 and 100°F) and were 4.2 and 4.4 percent, respectively (Fig 27). The optimum asphalt contents for the Lubbock limestone mixture ranged from 7. 1 percent at 24°C (75°F) to about 7.3 percent at 10°C (50°F) (Fig 28). For the Lufkin sand mixture the range was from 5.7 percent at 38°C (100°F) to 6.0 percent at 24°C (75°F) (Fig 29).

The optimum asphalt contents for maximum resilient modulus of elasticity ranged from zero to 1.8 percentage points less than the laboratory AVR optimum. However, for the Eagle Lake and Lubbock mixtures combined, the range of optimum asphalt contents was zero to 0.3 percentage point less than the AVR optimum. For the Lufkin sand mixture the range was from 1.5 to 1.8 percentage points less than the AVR optimum.

There was little or no difference between the maximum value and the value at the AVR optimum in most cases, because the relationship between asphalt content and resilient modulus of elasticity generally tended to be poorly defined or flat except at low temperatures. Therefore, it was felt that asphalt content, within the range of typical design values, had little effect on resilient modulus of elasticity.

Permanent Deformation

The analysis of permanent deformation was limited since normalization of the applied stress was different for the various mixtures and test conditions. Since the relationship between permanent deformation and applied stress is not well established, it was not possible to obtain permanent deformation information for the same stress conditions. Therefore, the analysis primarily involved comparing the optimum asphalt contents for maximum resistance to permanent deformation to the AVR design optimum asphalt content.

The parameter used to analyze permanent deformation was the permanent vertical deformation per cycle. Using this parameter, Adedimila and Kennedy (Ref 3) and Brown and Snaith (Ref 12) have found an optimum asphalt content

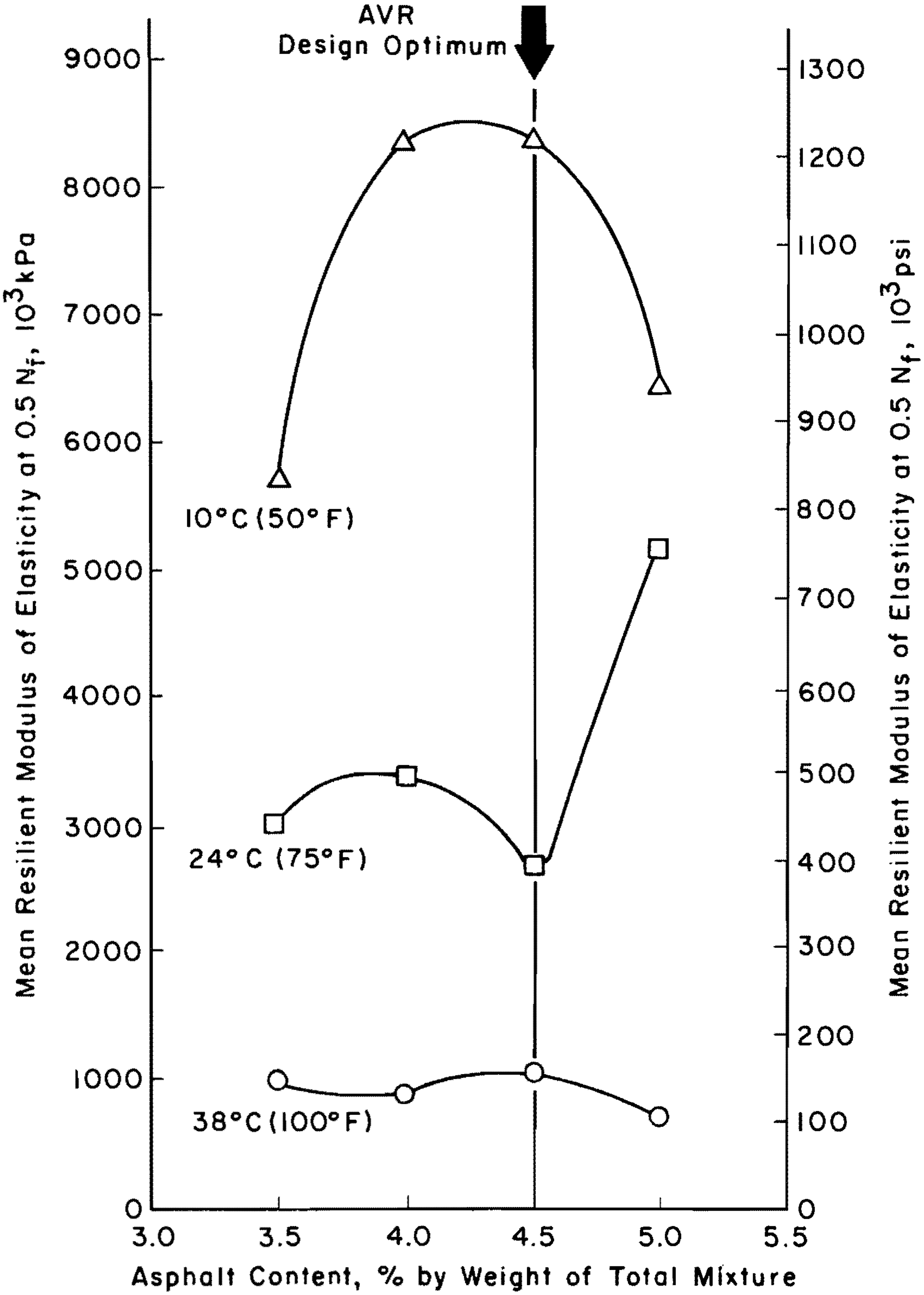


Fig 27. Effect of asphalt content on resilient modulus of elasticity for Eagle Lake gravel mixtures.

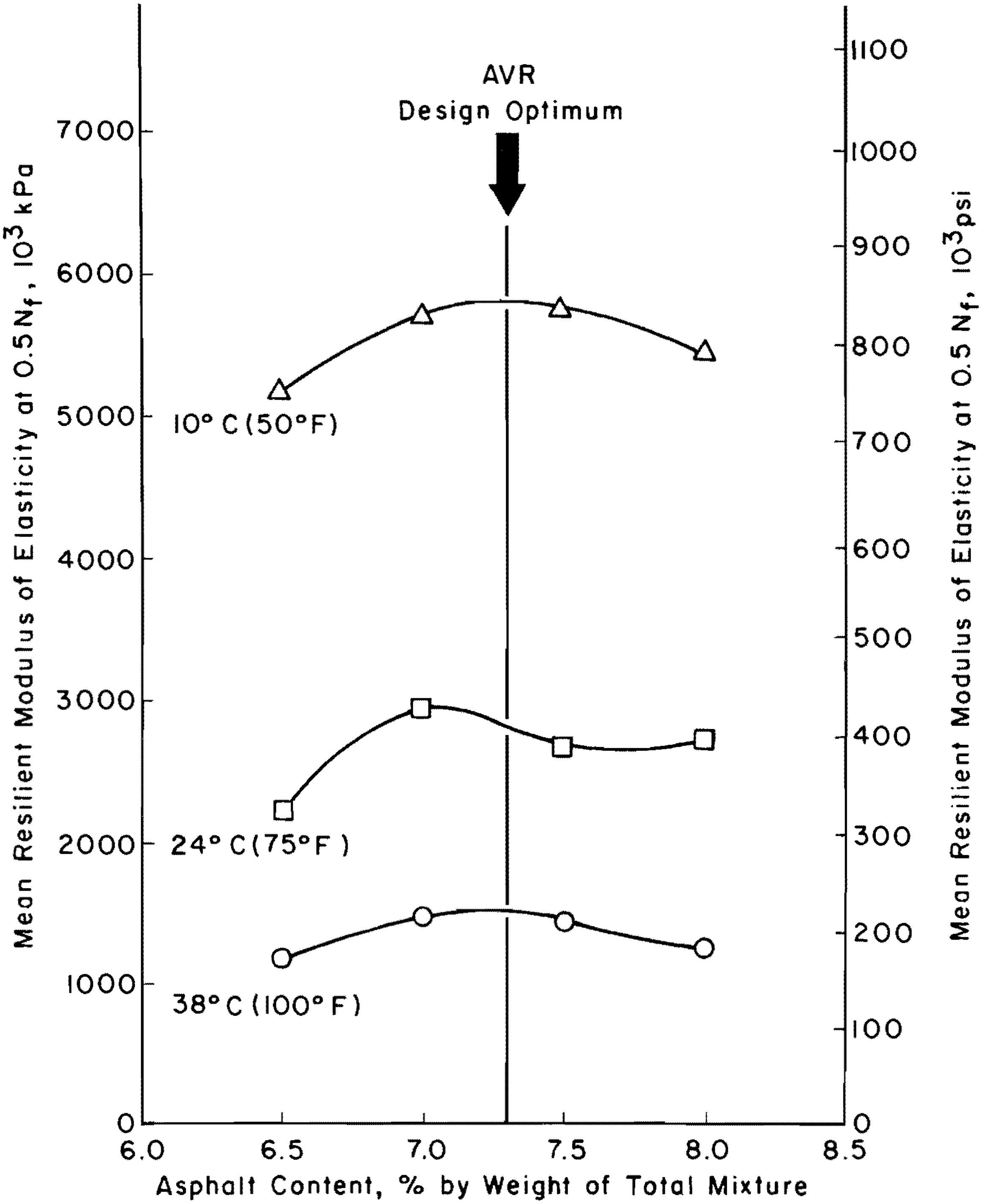


Fig 28. Effect of asphalt content on resilient modulus of elasticity for Lufkin sand mixtures.

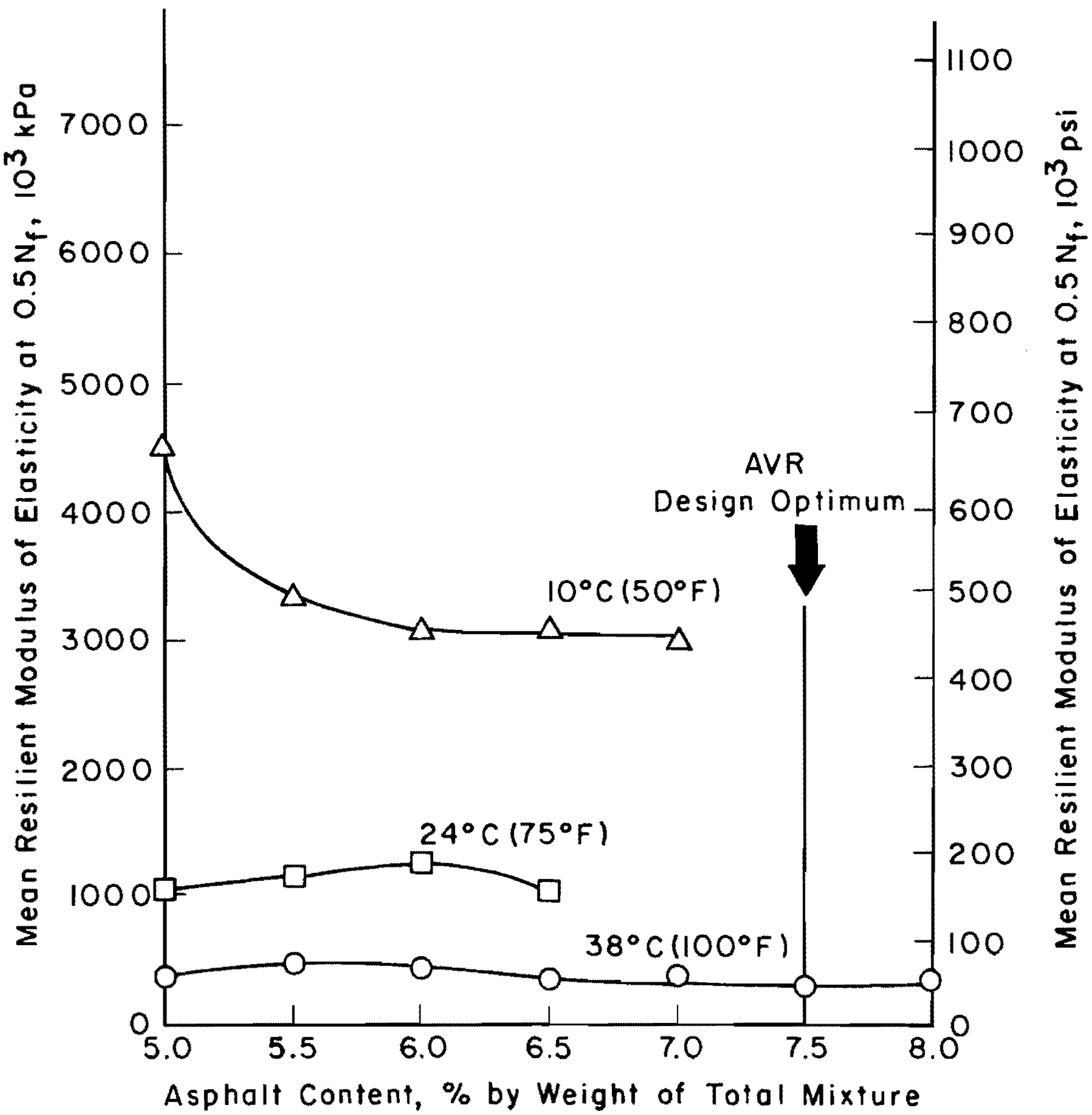


Fig 29. Effect of asphalt content on resilient modulus of elasticity for Lufkin sand mixtures.

for maximum resistance to permanent deformation. Values of permanent deformation for individual specimens in this study are shown in Appendix G.

For the Eagle Lake gravel mixture the optimum asphalt content for maximum resistance to permanent deformation ranged from 4.0 to 4.5 percent (Fig 30); for the Lubbock limestone mixture the range was from 7.1 to 7.4 percent (Fig 31); and for the Lufkin sand mixture the range was from 5.3 to 6.5 percent (Fig 32).

The optimum asphalt contents for maximum resistance to permanent deformations were from 0.1 percentage point greater to 2.2 percentage points less than the AVR design optimum. Except for one test condition, the optimums for the Eagle Lake gravel and Lubbock limestone mixtures ranged from zero to 0.5 percentage point less than the AVR design optimum. For the Lufkin sand mixture alone, the optimums were from 1.0 to 2.2 percentage points less than the AVR design optimum. Thus, the maximum resistance to permanent deformation usually occurred at asphalt contents below the AVR design optimum. For the Eagle Lake gravel and Lufkin sand mixtures the rate of increase in permanent deformations was larger on the wet side of the optimum asphalt content than on the dry side. The results for the Lubbock limestone mixture were not consistent.

Although no definite trends were observed, it was found that temperature and stress level both influenced the optimum asphalt content. In addition, it appears that the effect of asphalt content at high temperatures is greater than at low temperatures.

It should be mentioned that the intercept value (permanent vertical deformation at cycle number 1) for the linear logarithmic relationship between permanent vertical deformation and number of cycles did not appear to be useful for evaluating permanent deformation since it is highly dependent on seating errors. In addition, it did not correlate with the slope of permanent vertical deformation per cycle.

COMPARISON OF OPTIMUM ASPHALT CONTENTS

Test results indicated that optimum asphalt contents existed for various engineering properties, i.e., indirect tensile strength, static modulus of elasticity, fatigue life, minimum permanent deformation, and, to a certain extent, resilient modulus of elasticity. These optimums were different,

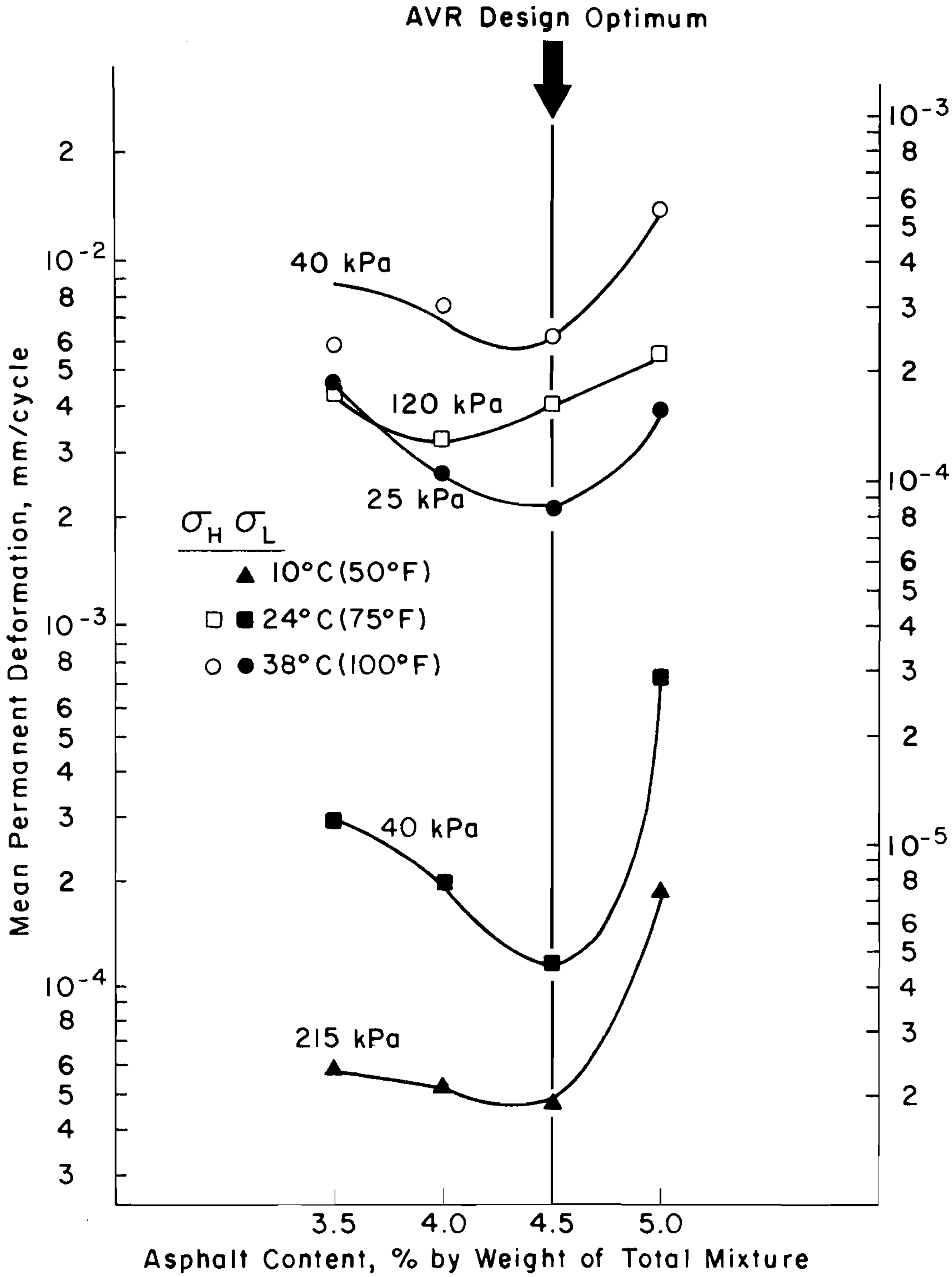


Fig 30. Effect of asphalt content and temperature on permanent deformation for Eagle Lake gravel mixtures.

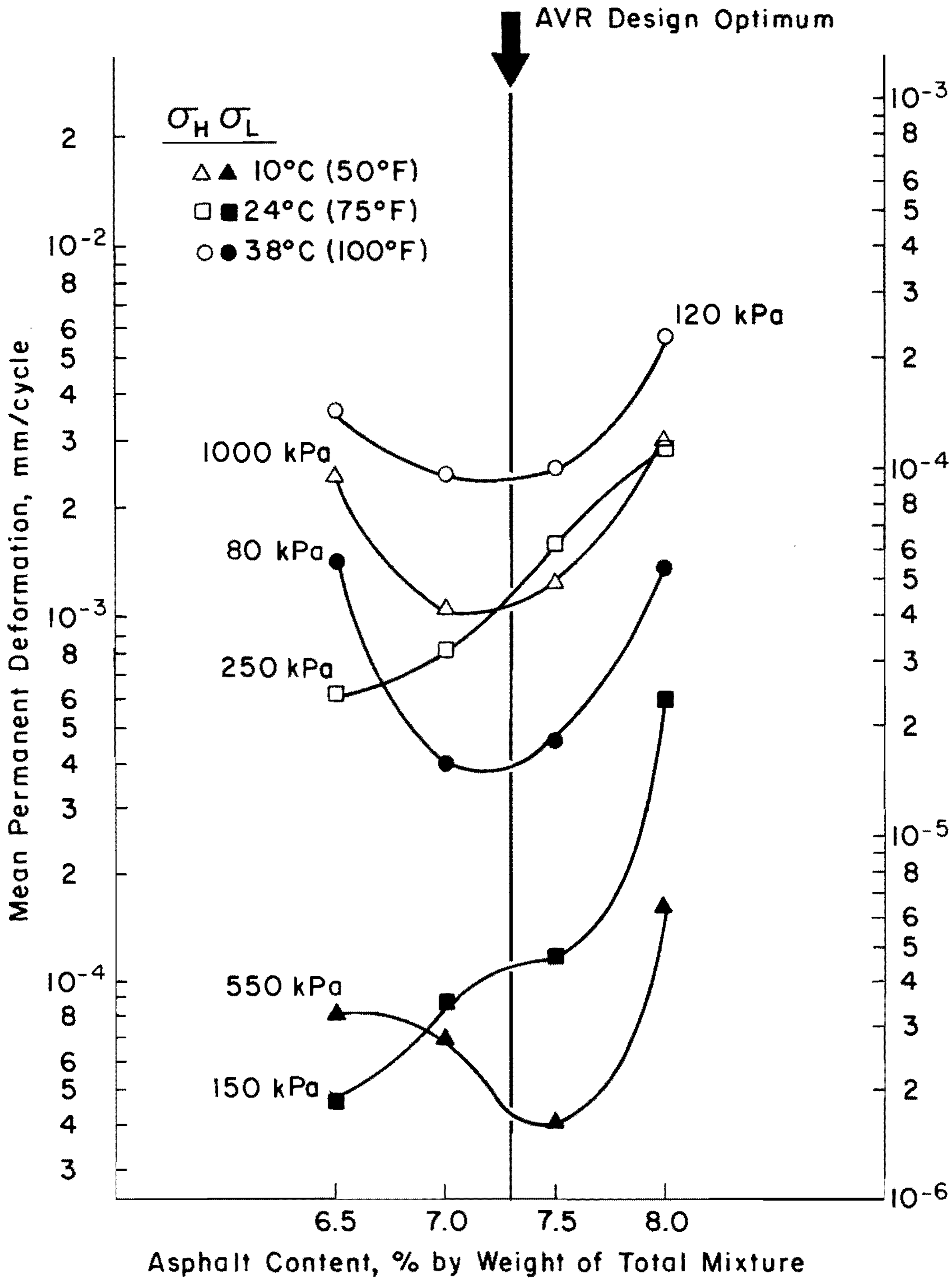


Fig 31. Effect of asphalt content and temperature on permanent deformation for Lubbock limestone mixtures.

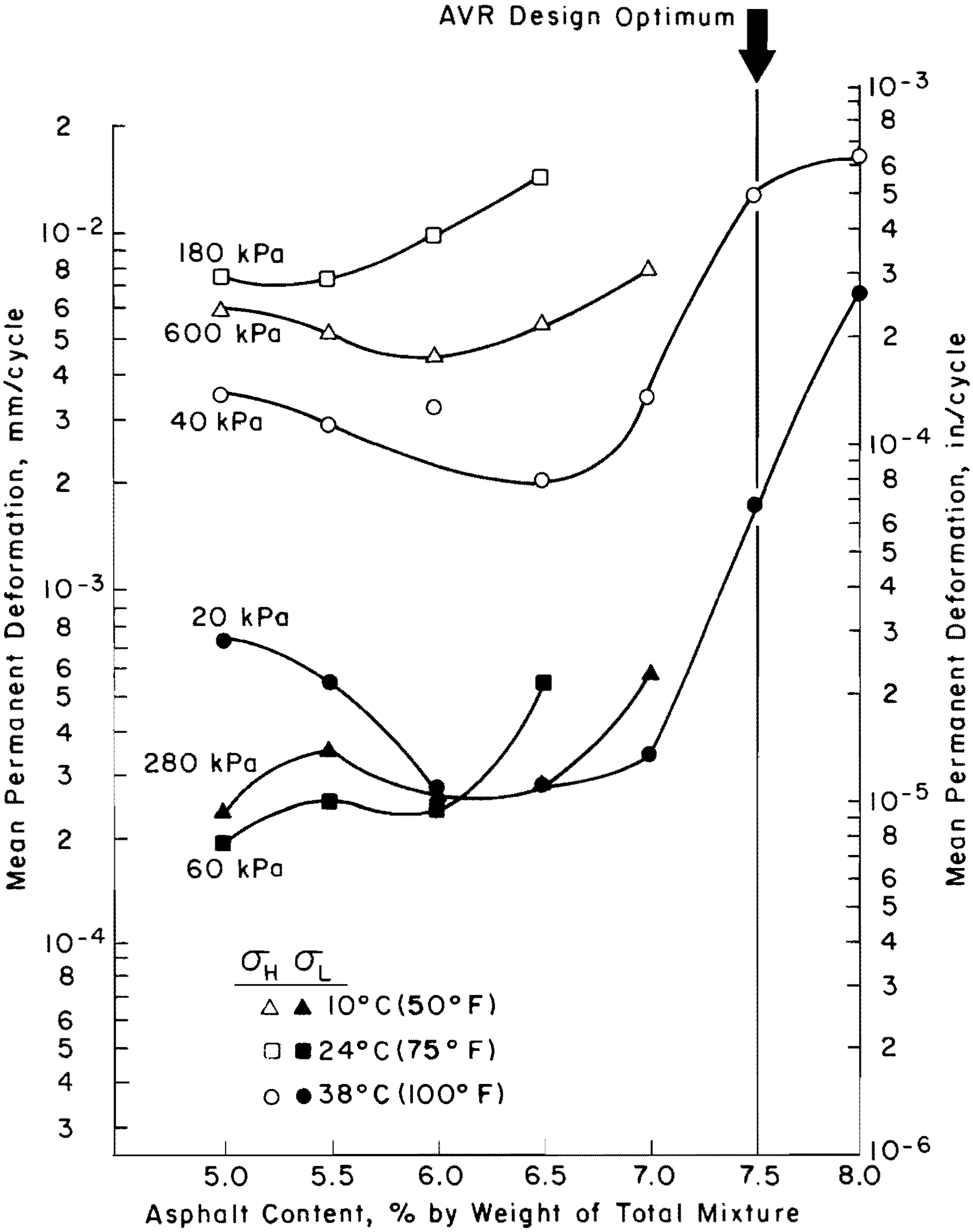


Fig 32. Effect of asphalt content and temperature on permanent deformation for Lufkin sand mixtures.

however, and in addition were not the same as the AVR design optimum or the optimum for in-mold AVR density.

The relationships between optimum asphalt contents and test temperature for these properties are shown in Figs 33, 34, and 35 for the Eagle Lake, Lubbock, and Lufkin mixtures, respectively. For comparison, the laboratory AVR design asphalt content and the optimum asphalt content for maximum in-mold AVR density are also shown.

Several general trends were observed in all three materials.

(1) The selected AVR design optimum was approximately 0.3 percentage point less than the optimum for maximum in-mold AVR density except for the Lufkin sand mixture, for which the two optimums were equal.

(2) Except for tests at 24°C (75°F) for the Eagle Lake gravel mixture and fatigue tests for the Lubbock limestone mixture, the optimum asphalt contents for all properties were less than the AVR design optimum.

(3) The optimums for the static properties were less than those for the repeated-load properties.

    (a) The optimum asphalt contents for static tensile properties of the Eagle Lake gravel and Lubbock limestone mixtures were from 0.1 to 1.2 percentage points less than the AVR design optimum.

    (b) The optimums for the Lufkin sand mixture were from 1.5 to 2.7 percentage points less than the AVR design optimum.

(4) The optimum asphalt contents for static modulus of elasticity were the smallest of the optimums identified.

(5) The optimum asphalt contents for tensile strength occurred at slightly richer asphalt contents at 24 and 38°C (75 and 100°F) than did the optimums for static modulus of elasticity.

(6) The optimum asphalt contents for fatigue life generally were larger than the optimums for the other material properties studied although this trend was not as strong in the Eagle Lake gravel mixture as in the other mixtures.

(7) The optimum asphalt contents for resistance to permanent deformation and instantaneous resilient modulus of elasticity generally were greater than the optimums for static properties but less than the optimums for fatigue life.

From the preceeding discussion and from Figs 33, 34, and 35, it may be concluded that the optimum asphalt content for static and repeated-load properties is generally less than the optimum asphalt content obtained by using Test Method Tex-126-E. These test results indicate that for the engineering properties discussed herein, optimum performance for various properties will be

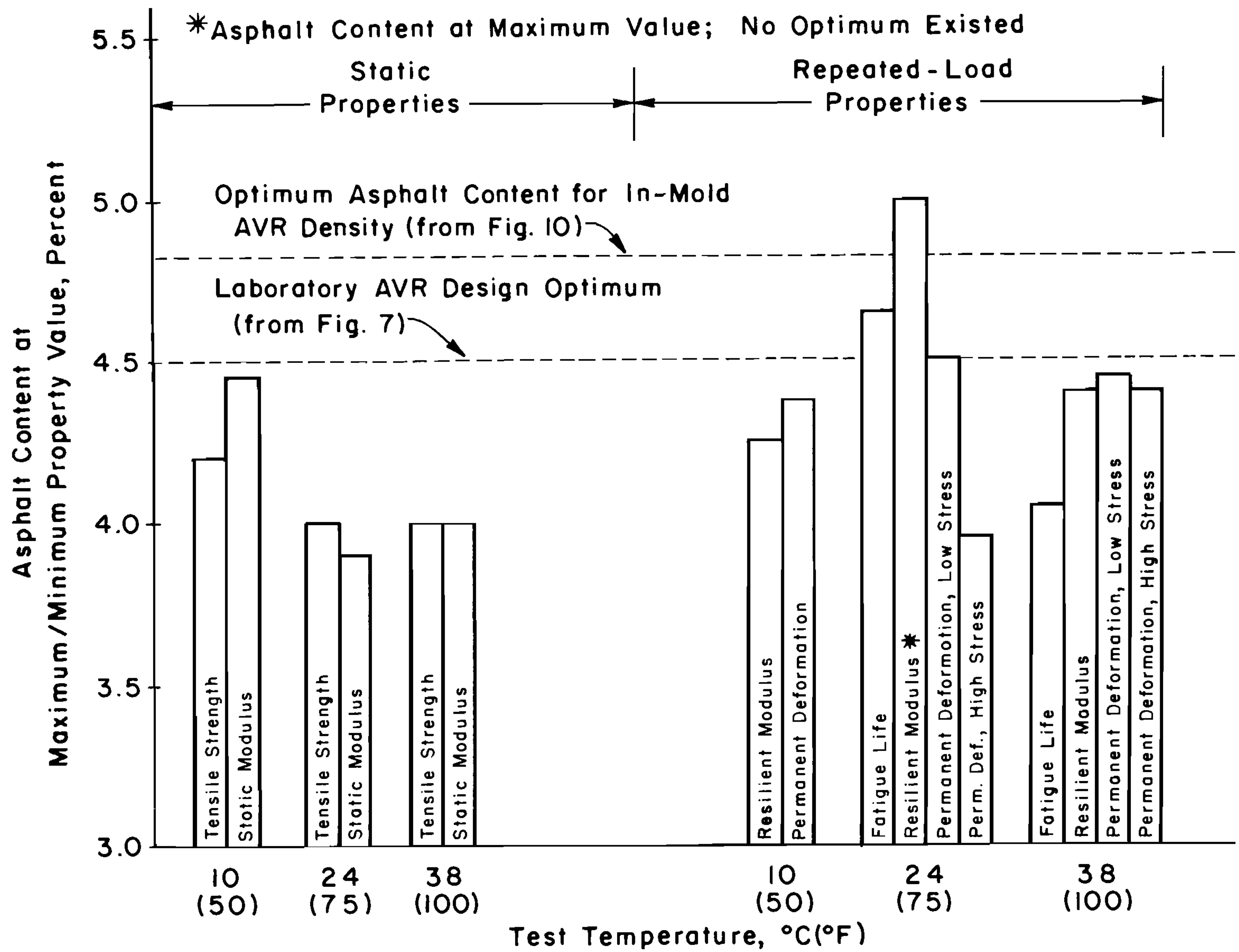


Fig 33. Relationship between testing temperature and the optimum asphalt contents for engineering properties of Eagle Lake gravel mixtures.

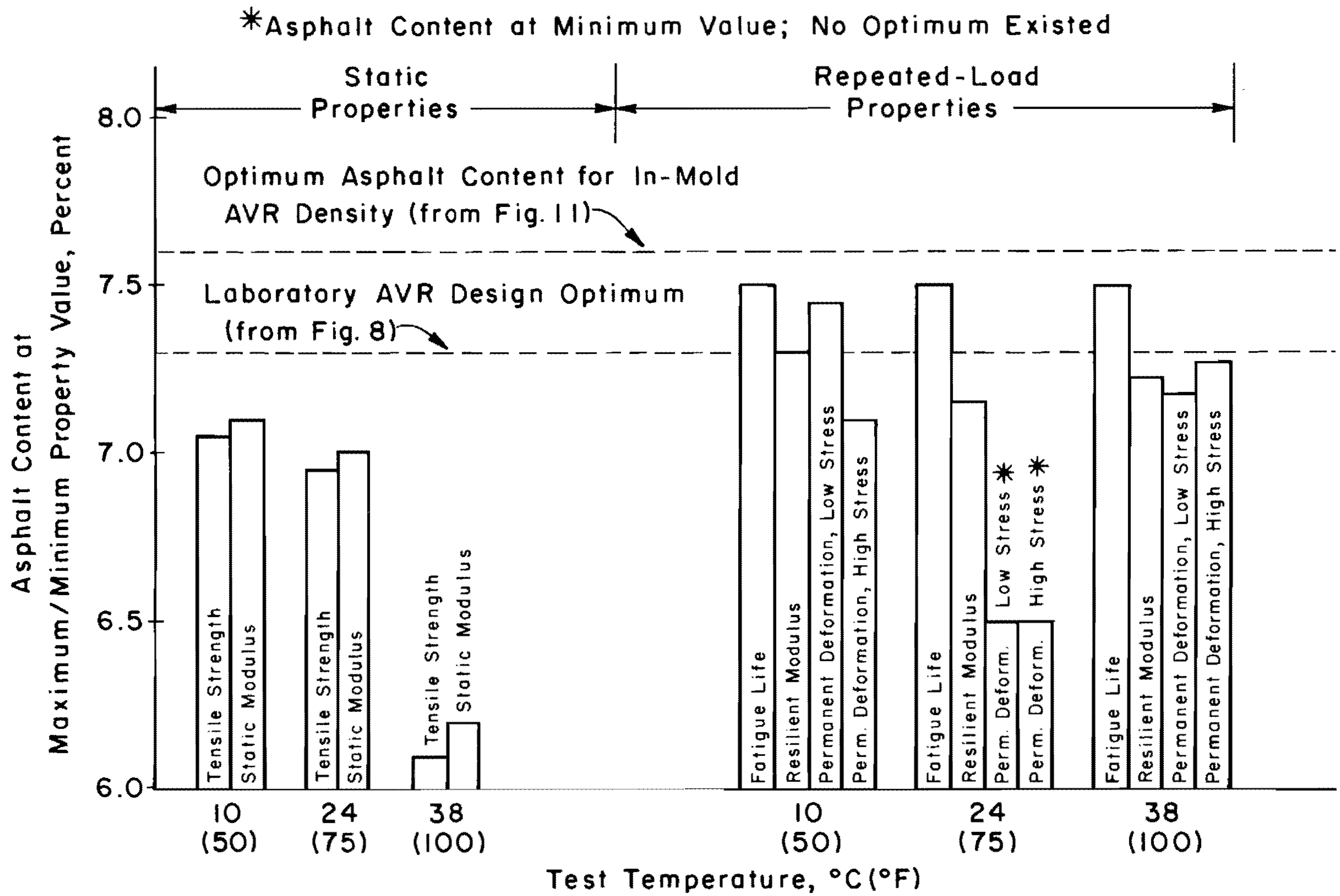


Fig 34. Relationship between testing temperature and optimum asphalt contents for engineering properties of Lubbock limestone mixtures.

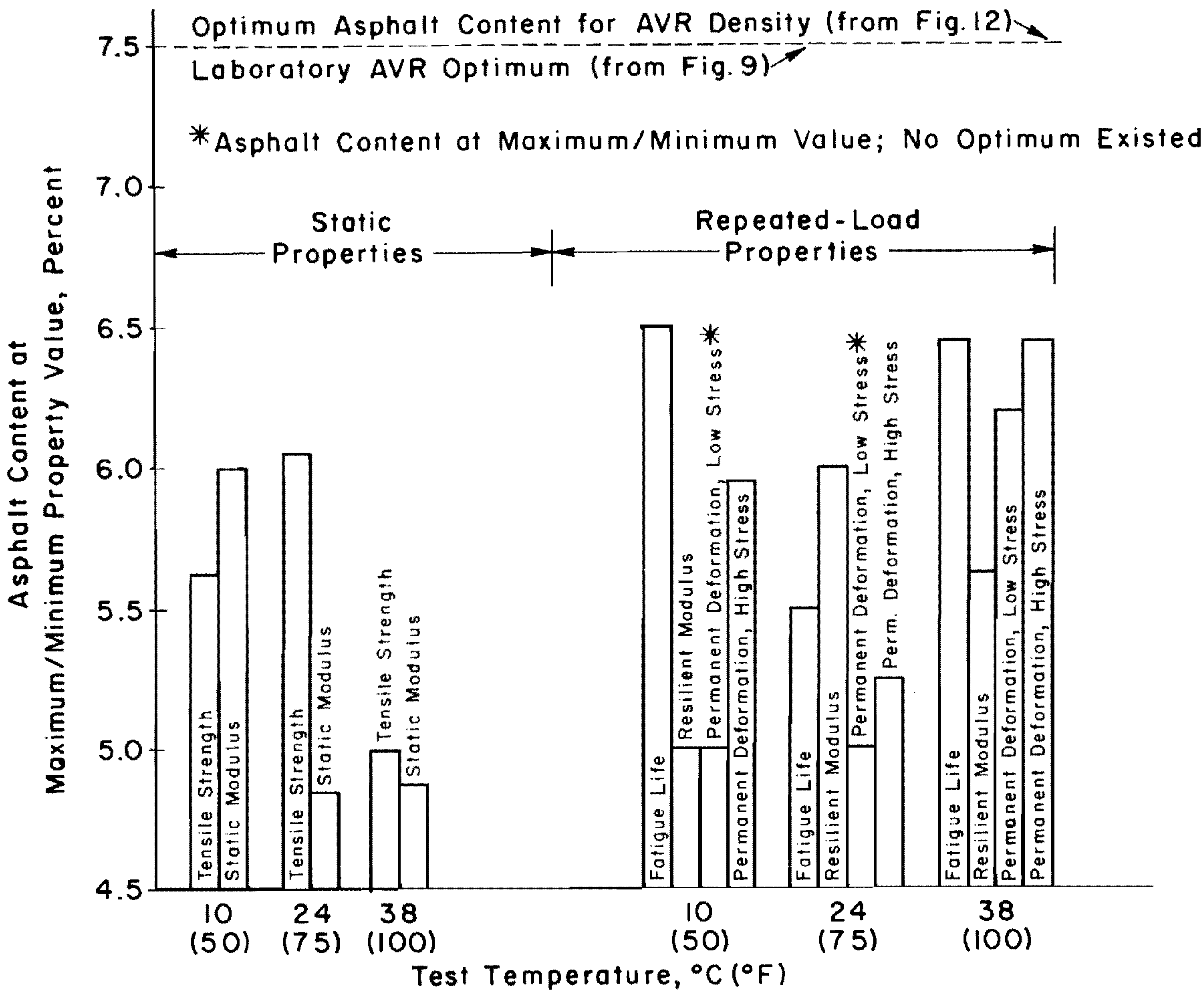


Fig 35. Relationship between testing temperature and optimum asphalt contents for engineering properties of Lufkin sand mixtures.

found generally at asphalt contents less than the design optimum asphalt content, depending upon the property under consideration and the aggregate. However, an immediate question may arise regarding the effect of moisture at these asphalt contents.

The effect of moisture at these lower asphalt contents was not considered in this investigation but may have a significant effect on the performance of an asphalt mixture with reduced asphalt contents. Consequently, a study evaluating the effects of moisture at these asphalt contents should be undertaken before making a judgment on the field performance of these materials at lower asphalt contents than the design optimum asphalt content.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

Within the limits of load, asphalt contents, mixture, and temperature variables considered in this study, the following conclusions and recommendations are made.

CONCLUSIONS

General

(1) The AVR design optimum asphalt contents generally was higher than the optimum asphalt contents for the engineering material properties of tensile strength, static modulus of elasticity, resilient modulus, fatigue life, and permanent deformation characteristics as measured using the static and repeated-load indirect tensile test.

(2) It would appear, based on information supplied by DHT, that Test Method Tex-126-E does not consistently describe the pavement performance of an asphalt mixture.

(3) Optimum asphalt contents were found to occur for the following material properties:

  (a) tensile strength,

  (b) static modulus of elasticity,

  (c) fatigue life, and

  (d) permanent deformation.

  Well defined optimums did not consistently occur for resilient modulus except at low temperatures.

(4) Generally, the optimum asphalt contents for static tensile properties were less than the optimums for the repeated-load properties.

  (a) The optimum for static modulus of elasticity was generally less than the optimum for tensile strength.

  (b) The optimum for fatigue life was larger than the optimums for the other engineering properties.

(c) The optimums for permanent deformation and instantaneous resilient modulus of elasticity were generally less than the optimum for fatigue life and larger than the optimum for static tensile properties.

(5) The static and repeated-load indirect tensile tests can be used to evaluate materials for mix design purposes.

Static Characteristics

(1) Asphalt content is very important with respect to tensile strength and static modulus of elasticity at low temperatures. The effect of asphalt content on tensile strength and static modulus of elasticity is not as important at higher temperatures.

(2) The optimum asphalt contents for tensile strength and static modulus of elasticity increased as temperature decreased.

Repeated-Load Characteristics

(1) Resilient modulus of elasticity also showed a reduced effect of asphalt content at higher temperatures but the effect was smaller than for tensile strength or static modulus of elasticity.

(2) In general, the effect of asphalt content on resilient modulus of elasticity was small.

(3) For the Lubbock limestone mixture, the optimum asphalt content for maximum fatigue life was essentially independent of temperature. For the Lufkin sand mixture, temperature did have an effect on the optimum for maximum fatigue life; the optimum at 24°C (75°F) was lower than the optimums at the other temperatures.

(4) It was found that the rate of increase in permanent deformation was slightly greater wet of optimum asphalt content than dry of optimum. All other properties were essentially the same wet and dry of optimum.

RECOMMENDATIONS

(1) It is recommended that static indirect tensile tests be performed on all blackbase mixtures as part of the current mixture design procedure. If possible, repeated-load indirect tensile tests should also be conducted. Initially repeated-load tests should be

conducted by the Materials and Tests Division; however, most of the district laboratories can conduct the static tests to measure strength. It is recommended that a test procedure for the indirect tensile test be included as a part of the correct mixture design procedure.

(2) Static and repeated-load indirect tensile tests should be used as a check for the optimum asphalt content found by Test Method Tex-126-E. These test results can be used to help establish the final optimum design asphalt content, but probably should not be used as the only criterion if large changes are indicated.

(3) When possible, static and repeated-load indirect tensile tests should be performed on specimens of field mixtures. Specimens ideally should be prepared at the plant to avoid reheating the mixture.

Future Studies

(1) Further studies should be conducted to establish design criteria using the indirect tensile test.

(2) Studies to investigate the effects of moisture damage should be conducted and subsequently incorporated into the mixture design procedure.

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APPENDIX A

WASHED GRADATION OF COMBINED AGGREGATES

APPENDIX A. WASHED GRADATION OF COMBINED AGGREGATES

Sieve Size	Cumulative Percent Retained		
	Eagle Lake Gravel	Lubbock Limestone	Lufkin Sand
+ 1-3/4"			
+ 1-1/2"			
+ 1-1/4"	3.4		
+ 1	15.0		
+ 7/8"	19.0	12.1	
+ 5/8"	27.0	25.6	
+ 1/2"	31.6	33.1	
+ 3/8"	37.0	44.0	
+ # 4	51.4	57.6	
+ # 10	59.0	65.9	0.9
+ # 20	63.0		
+ # 40	70.0	72.4	31.6
+ # 80	91.0	86.3	72.3
+ #200	99.0	92.3	81.5
Specific Gravity	2.63	2.68	2.64

APPENDIX B

WASHED AGGREGATE GRADATIONS OF

AGGREGATE COMPONENTS

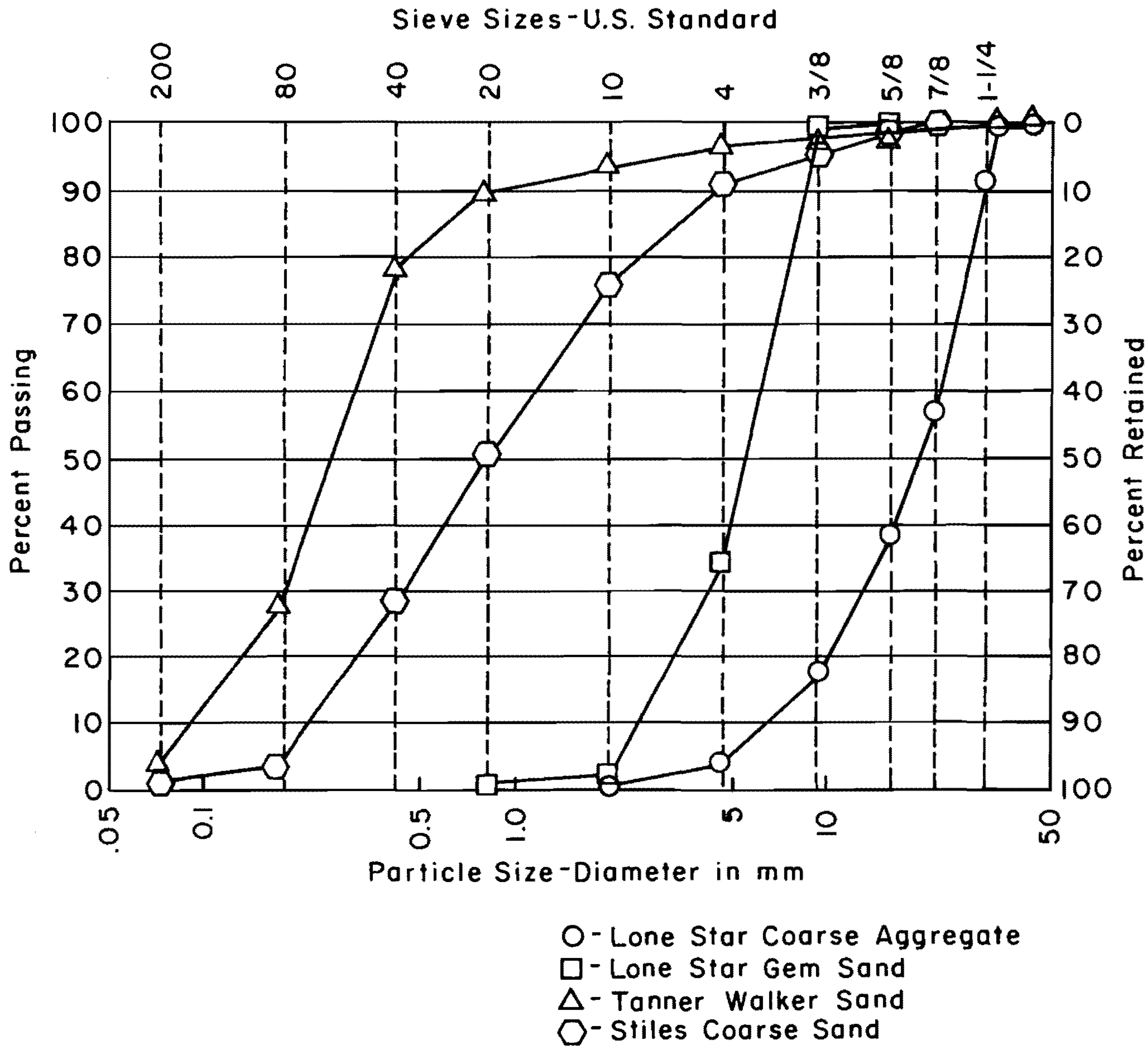

Fig B-1. Washed aggregate gradation for Eagle Lake gravel components.

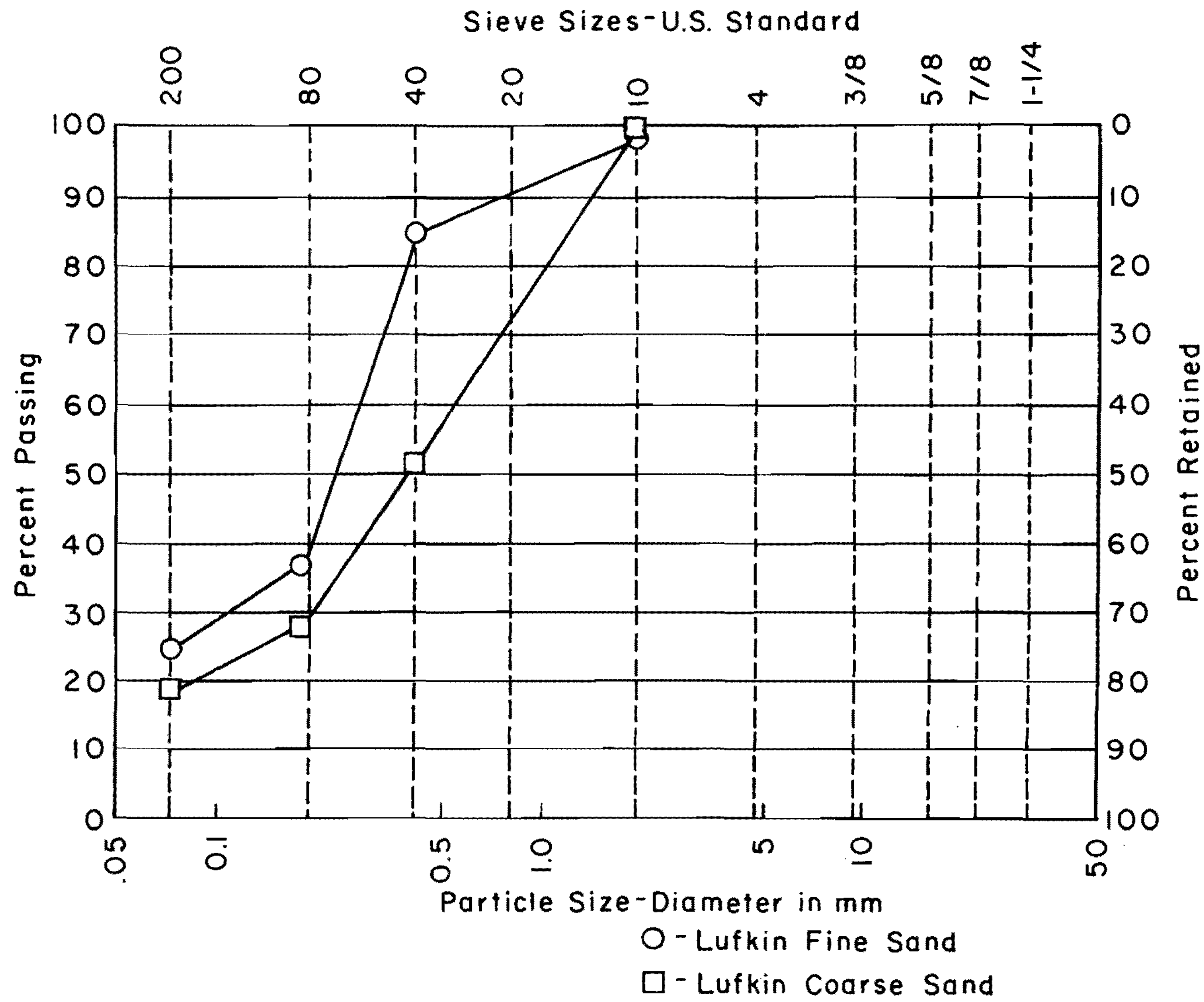

Fig B-2. Washed aggregate gradation for Lufkin sand components.

APPENDIX C

SPECIMEN PREPARATION

APPENDIX C. SPECIMEN PREPARATION

Mixing

(1) The aggregate was batched by dry weight for at least seven sieve sizes. However, grading of the Lufkin sand was not necessary since segregation was not a problem.

(2) The aggregate was divided into plus No. 10 and minus No. 10 portions (except for the Lufkin sand), and heated to 177 $\pm$ 10°C (350 $\pm$ 20°F).

(3) The asphalt, heated to 115°C (240°F) was added to the plus No. 10 portion and mixed in an 11-liter (12-quart) capacity Hobart Automatic Mixer until well coated.

(4) The minus No. 10 portion was added to the plus No. 10 and asphalt mixture and was mixed until completely coated.

Compaction

(1) The 305 by 152 mm (12 by 6 inch) mold and base plate were preheated to 110°C (230°F) to maintain heat in the uncompacted specimen.

(2) The aggregate and asphalt mixtures were placed into a square mixing pan over a hotplate after mixing was completed. The large stones in the mixtures were then evenly distributed into the four corners of the mixing pan.

(3) The mold was loaded in four layers. First, the bottom of the mold was covered with about 12 mm (1/2 inch) of fines. The mold was then alternately loaded with large stones and fine material. Each layer was tamped with a trowel. The fourth layer was topped by about 6 mm (1/4 inch) of fines.

(4) The temperature after loading the mold was 127 $\pm$ 10°C (260 $\pm$ 20°F).

(5) The mold was then placed in the Texas Motorized Gyratory Press.

(6) With the mold at a 5° lift angle, a 150 kPa (20 psi) compressive load was placed on the specimen. The mode was gyrated in this condition for two minutes.

(7) The load was increased to 276 kPa (40 psi) and gyrated for two additional minutes.

(8) The compressive load on the specimen was then increased to 414 kPa (60 psi) and gyrated until the vertical deformation was 0.025 mm (0.001 inch) or less for five revolutions.

(9) The load was removed completely and the lift angle was returned to zero. The specimen was then loaded again to 150 kPa (20 psi) and rotated for several revolutions to square the ends of the specimen.

(10) A 3450 kPa (500 psi) compressive load was placed on the specimen. This load was maintained until the deformation was 0.127 mm (0.005 inch) or less in five minutes.

(11) The specimen height was measured while the specimen was in the mold. This height was used for AVR density and air void determinations.

All specimens were prepared identically up to this point. The remaining steps in specimen preparation are described in the text of Chapter 2.

APPENDIX D

IN-MOLD AVR DENSITY

TABLE D-1. IN-MOLD AVR DENSITY FOR EAGLE LAKE GRAVEL MIXTURES

	Asphalt Content, % by Weight of Total Mixture				
	3.5	4.0	4.5	5.0	5.5
Density, kg/m^3	2,358	2,355	2,390	2,404	2,390
	2,364	2,383	2,404	2,398	2,387
	2,363	2,380	2,398	2,400	
	2,343	2,382	2,404	2,398	
	2,342	2,368	2,387	2,387	
	2,356	2,387	2,390	2,390	
	2,356	2,379	2,388	2,396	
	2,355	2,395	2,396	2,396	
	2,350	2,366	2,396	2,388	
	2,360	2,377	2,396	2,395	
	2,353	2,384	2,390	2,392	
		2,374	2,387	2,387	
		2,366	2,393		
			2,380		
			2,382		
Average AVR density kg/m^3	2,355	2,377	2,392	2,395	2,388
(pcf)	(147.0)	(148.4)	(149.3)	(149.5)	(149.1)
Average total air voids, %	5.3	3.7	2.4	1.6	1.1

TABLE D-2. IN-MOLD AVR DENSITY FOR LUBBOCK LIMESTONE MIXTURES

	Asphalt Content, % by Weight of Total Mixtures						
	5.5	6.0	6.5	7.0	7.5	8.0	8.5
	2,102	2,156	2,202	2,185	2,206	2,185	2,196
		2,130	2,187	2,206	2,191	2,102	2,196
			2,145	2,202	2,198	2,206	
			2,148	2,219	2,203	2,187	
			2,146	2,180	2,220	2,190	
			2,162	2,198	2,203	2,195	
			2,138	2,182	2,198	2,193	
Density,			2,190	2,174	2,187	2,172	
kg/m^3			2,132	2,175	2,196	2,196	
			2,145	2,188	2,188	2,180	
				2,201	2,191		
				2,187	2,193		
				2,177	2,193		
				2,177	2,183		
				2,164			
				2,180			
				2,172			
Average AVR density, kg/m^3	2,102	2,143	2,159	2,187	2,196	2,188	2,196
(pcf)	(131.2)	(133.8)	(134.8)	(136.5)	(137.1)	(136.6)	(137.1)
Average total air voids, %	14.6	12.3	11.0	9.2	8.1	7.8	6.8

TABLE D-3. IN-MOLD AVR DENSITY FOR LUFKIN SAND MIXTURES

	Asphalt Content, % by Weight of Total Mixture								
	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5
	2,275	2,249	2,267	2,291	2,289	2,289	2,327	2,313	2,297
	2,231	2,252	2,289	2,302	2,291	2,331	2,325		
	2,223	2,236	2,255	2,278	2,270	2,331	2,324		
	2,207	2,252	2,254	2,281	2,279	2,276			
		2,255	2,231	2,260	2,291	2,337			
Density, kg/m^3		2,231	2,239	2,275	2,302	2,335			
		2,233	2,236	2,268	2,318				
		2,236	2,257	2,270	2,313				
		2,230	2,236	2,283	2,343				
		2,246	2,278	2,278	2,345				
		2,286	2,267	2,331					
		2,286	2,247	2,297					
			2,273						
Average AVR density, kg/m^3	2,235	2,249	2,255	2,284	2,303	2,316	2,326	2,313	2,297
(pcf)	(139.5)	(140.4)	(140.8)	(142.6)	(143.8)	(144.6)	(145.2)	(144.4)	(143.4)
Average total air voids, %	9.2	7.9	7.0	5.1	3.6	2.4	1.3	1.2	1.1

APPENDIX E

SPECIMEN PROPERTIES AND

STATIC TEST RESULTS

TABLE E-1. SPECIMEN PROPERTIES AND STATIC TEST RESULTS FOR EAGLE LAKE GRAVEL MIXTURES

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
10 (50)	3.5	2,329 (145.4)	6.4	2,930 (425)	3,173,000 (460,100)	-0.10
	3.5	2,311 (144.3)	7.1	2,320 (337)	4,588,000 (665,200)	0.00
	4.0	2,369 (147.9)	4.0	3,090 (448)	3,908,000 (566,600)	0.04
	4.0	2,353 (146.9)	4.6	3,100 (449)	3,500,000 (507,500)	-0.02
	4.5	2,345 (146.4)	4.3	3,230 (468)	4,097,000 (594,200)	-0.16
	4.5	2,377 (148.4)	3.0	2,900 (421)	4,988,000 (723,200)	-0.02
	5.0	2,339 (146.0)	3.9	2,780 (403)	3,019,000 (437,800)	-0.05
	5.0	2,348 (146.6)	3.5	2,940 (427)	2,509,000 (363,800)	0.10
24 (75)	3.5	2,332 (145.6)	6.3	1,080 (156)	1,900,000 (275,500)	0.50
	3.5	2,324 (145.1)	6.5	880 (128)	1,810,000 (262,400)	0.45
	4.0	2,363 (147.5)	4.3	1,250 (181)	2,606,000 (377,800)	0.39
	4.0	2,363 (147.5)	4.3	1,120 (163)	2,081,000 (301,800)	0.61
	4.5	2,366 (147.7)	3.4	990 (144)	1,112,000 (161,200)	0.35
	4.5	2,382 (148.7)	2.8	880 (128)	995,900 (144,400)	0.37
	5.0	2,335 (145.8)	4.0	740 (108)	635,200 (92,100)	-0.02
	5.0	2,348 (146.6)	3.5	740 (108)	512,000 (74,240)	0.26

(continued)

TABLE E-1. (continued)

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
38 (100)	3.5	2,337 (145.9)	6.1	280 (40)	503,800 (73,050)	1.17
	3.5	2,318 (144.7)	6.8	200 (29)	245,900 (35,650)	0.89
	4.0	2,379 (148.5)	3.6	330 (48)	482,800 (70,010)	0.85
	4.0	2,369 (147.9)	4.0	300 (43)	412,100 (59,760)	0.83
	4.5	2,356 (147.1)	3.8	250 (36)	261,700 (37,950)	0.62
	4.5	2,374 (148.2)	3.1	280 (41)	403,000 (58,430)	0.82
	5.0	2,342 (146.2)	3.7	170 (25)	127,600 (18,500)	0.39
	5.0	2,356 (147.1)	3.1	170 (25)	103,030 (14,940)	0.33

TABLE E-2. SPECIMEN PROPERTIES AND STATIC TEST RESULTS FOR LUBBOCK LIMESTONE MIXTURES

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
	6.5	2,172 (135.6)	10.5	2,830 (411)	1,961,000 (284,400)	0.28
	6.5	2,183 (136.3)	10.0	2,740 (397)	2,059,000 (298,600)	0.01
	6.5	2,127 (132.8)	12.3	2,360 (342)	1,575,000 (228,400)	0.20
	6.5	2,129 (132.9)	12.2	2,270 (329)	1,703,000 (246,900)	0.09
	7.0	2,191 (136.8)	9.0	2,540 (368)	2,145,000 (311,100)	0.54
	7.0	2,198 (137.2)	8.7	2,360 (343)	1,995,000 (289,300)	0.21
	7.0	2,187 (136.5)	9.2	3,110 (451)	2,748,000 (398,400)	0.22
	7.0	2,199 (137.3)	8.6	2,960 (429)	2,974,000 (431,200)	0.34
10 (50)	7.0	2,172 (135.6)	9.8	2,810 (408)	2,405,000 (348,700)	-0.04
	7.0	2,185 (136.4)	9.2	2,660 (386)	2,675,000 (387,900)	0.04
	7.5	2,187 (136.5)	8.4	2,450 (355)	1,761,000 (255,300)	0.21
	7.5	2,182 (136.2)	8.7	2,210 (321)	2,654,000 (384,800)	-0.02
	7.5	2,195 (137.0)	8.1	2,800 (406)	2,256,000 (327,100)	0.30
	7.5	2,211 (138.0)	7.6	2,810 (407)	2,682,000 (388,900)	0.20
	7.5	2,201 (137.4)	8.0	2,630 (382)	2,748,000 (398,500)	0.08
	7.5	2,198 (137.2)	8.0	2,470 (359)	2,748,000 (398,400)	0.09

(continued)

TABLE E-2. (continued)

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m³ (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
10 (50)	8.0	2,187 (136.5)	7.8	2,180 (316)	1,706,000 (247,300)	0.17
	8.0	2,166 (135.2)	8.8	2,000 (290)	1,523,000 (220,800)	0.20
	8.5	2,174 (135.7)	7.7	1,990 (288)	1,459,000 (211,500)	0.04
	8.5	2,185 (136.4)	7.3	1,990 (289)	821,400 (119,100)	0.16
24 (75)	6.0	2,134 (133.2)	12.7	1,230 (179)	1,170,000 (169,600)	0.17
	6.0	2,147 (134.0)	12.1	1,190 (173)	997,900 (144,700)	0.24
	6.5	2,138 (135.5)	11.8	1,190 (173)	1,359,000 (197,000)	0.16
	6.5	2,142 (133.7)	11.7	1,120 (163)	942,000 (136,600)	0.41
	6.5	2,135 (133.3)	12.0	1,080 (157)	972,400 (141,000)	0.44
	6.5	2,158 (134.7)	11.1	1,140 (165)	981,400 (142,300)	0.36
	7.0	2,183 (136.3)	9.4	840 (122)	1,487,000 (215,600)	0.30
	7.0	2,177 (135.9)	9.6	720 (104)	1,069,000 (155,000)	0.26
	7.0	2,167 (135.3)	10.0	1,270 (184)	1,652,000 (239,600)	0.70
	7.0	2,166 (135.2)	10.0	1,210 (176)	1,288,000 (186,800)	0.21
	7.0	2,161 (134.9)	10.2	1,250 (181)	831,000 (120,800)	0.21
	7.0	2,172 (135.6)	9.8	1,190 (172)	915,200 (132,700)	0.39
	7.5	2,182 (136.2)	8.7	900 (131)	746,200 (108,200)	0.34
	7.5	2,204 (137.6)	7.8	860 (125)	712,400 (103,300)	0.30

(continued)

TABLE E-2. (continued)

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
24 (75)	7.5	2,180 (136.1)	8.8	970 (141)	1,278,000 (185,300)	0.37
	7.5	2,179 (136.0)	9.0	970 (141)	1,007,000 (146,000)	0.25
	8.0	2,190 (136.7)	7.8	700 (102)	952,400 (138,100)	0.18
	8.0	2,188 (136.6)	7.8	740 (107)	750,300 (108,800)	0.41
	8.5	2,179 (136.0)	7.5	570 (82)	400,100 (58,010)	0.19
	8.5	2,190 (136.7)	7.1	530 (77)	849,000 (123,100)	0.31
38 (100)	5.5	2,103 (131.3)	14.5	480 (70)	447,600 (64,900)	0.44
	5.5	2,058 (128.5)	16.3	340 (50)	331,600 (48,080)	0.56
	6.0	2,211 (138.0)	9.5	570 (82)	744,800 (108,000)	0.00
	6.0	2,193 (136.9)	10.2	460 (67)	587,000 (85,120)	0.32
	6.0	2,158 (134.7)	11.7	540 (78)	650,600 (94,330)	0.33
	6.0	2,167 (135.3)	11.3	500 (73)	736,600 (106,800)	0.32
	6.5	2,182 (136.2)	10.0	360 (52)	524,000 (75,980)	0.09
	6.5	2,177 (135.9)	10.2	380 (55)	541,900 (78,580)	0.31
	6.5	2,145 (133.9)	11.5	530 (77)	452,800 (65,650)	0.54
	6.5	2,166 (135.2)	10.7	530 (79)	587,800 (85,230)	0.15
	7.0	2,190 (136.7)	9.0	320 (47)	606,100 (87,890)	0.40
	7.0	2,206 (137.7)	8.4	340 (49)	513,200 (74,420)	0.43

(continued)

TABLE E-2. (continued)

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
38 (100)	7.0	2,170 (135.5)	9.8	540 (79)	691,700 (100,300)	0.40
	7.0	2,195 (137.0)	8.8	480 (70)	516,700 (74,920)	0.50
	7.5	2,187 (136.5)	8.5	280 (41)	363,700 (52,730)	0.30
	7.5	2,198 (137.2)	8.0	230 (34)	313,200 (45,410)	0.43
	8.0	2,190 (136.7)	7.7	240 (35)	320,900 (46,530)	0.37
	8.0	2,190 (136.7)	7.7	230 (34)	208,800 (30,270)	0.28

TABLE E-3. SPECIMEN PROPERTIES AND STATIC TEST RESULTS FOR LUFKIN SAND MIXTURES

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
10 (50)	4.5	2,241 (139.9)	8.9	1,660 (240)	1,317,000 (191,000)	0.52
	4.5	2,214 (138.2)	10.0	1,490 (216)	894,000 (129,600)	0.35
	5.0	2,246 (140.2)	8.1	1,810 (263)	1,156,000 (167,600)	0.32
	5.0	2,220 (138.6)	9.1	1,630 (236)	1,085,000 (157,300)	0.34
	5.5	2,270 (141.7)	6.4	1,970 (285)	1,228,000 (178,000)	0.37
	5.5	2,238 (139.7)	7.7	1,700 (246)	940,000 (136,300)	0.33
	6.0	2,276 (142.1)	5.5	1,740 (252)	1,429,000 (207,200)	0.35
	6.0	2,299 (143.5)	4.5	1,870 (271)	1,803,000 (261,500)	0.18
	6.5	2,292 (143.1)	4.1	1,720 (250)	1,014,000 (147,100)	0.35
	6.5	2,268 (141.6)	5.1	1,560 (226)	906,000 (131,400)	0.36
24 (75)	4.5	2,217 (138.4)	9.9	630 (92)	386,100 (55,980)	0.66
	4.5	2,180 (136.1)	11.4	610 (88)	317,100 (45,980)	0.74
	5.0	2,215 (138.3)	9.3	710 (103)	409,100 (59,320)	0.63
	5.0	2,196 (137.1)	10.1	670 (97)	373,700 (54,180)	0.59
	5.0	2,212 (138.1)	9.4	590 (86)	297,300 (43,110)	0.57
	5.5	2,227 (139.0)	8.2	700 (102)	366,000 (53,070)	0.72

(continued)

TABLE E-3. (continued)

Temperature, °C (°F)	Asphalt Content, %	Density, kg/m³ (pcf)	Air Void Content, %	Tensile Strength, kPa (psi)	Static Modulus of Elasticity E_S, kPa (psi)	Poisson's Ratio
24 (75)	5.5	2,219 (138.5)	8.5	670 (97)	335,200 (48,610)	0.71
	5.5	2,217 (138.4)	[illegible]	570 (83)	244,600 (35,480)	0.61
	6.0	2,241 (139.9)	6.9	720 (104)	361,700 (52,440)	0.75
	6.0	2,234 (139.5)	7.2	700 (102)	354,900 (51,460)	0.76
	6.0	2,263 (141.3)	6.0	650 (94)	278,900 (40,440)	0.54
	6.5	2,276 (142.1)	4.8	650 (94)	271,900 (39,430)	0.54
38 (100)	4.5	2,207 (137.8)	10.3	270 (39)	111,200 (16,120)	0.94
	4.5	2,191 (136.8)	10.9	260 (37)	124,900 (18,110)	0.94
	5.0	2,230 (139.2)	8.7	320 (46)	126,760 (18,380)	0.87
	5.0	2,265 (141.4)	7.3	300 (44)	111,100 (16,110)	0.95
	5.5	2,246 (140.2)	7.4	300 (44)	113,650 (16,480)	0.94
	5.5	2,235 (139.5)	7.9	290 (42)	95,170 (13,800)	0.80
	5.5	2,225 (138.9)	8.3	250 (36)	102,000 (14,790)	0.93
	6.0	2,249 (140.4)	6.6	300 (44)	103,100 (14,950)	0.95
	6.0	2,247 (140.3)	6.7	290 (42)	104,550 (15,160)	0.88
	6.0	2,267 (141.5)	5.8	260 (37)	92,480 (13,410)	0.90
	6.5	2,273 (141.9)	4.9	230 (34)	80,280 (11,640)	0.87

APPENDIX F

SPECIMEN PROPERTIES AND REPEATED-LOAD TEST RESULTS

TABLE F-1. SPECIMEN PROPERTIES AND REPEATED-LOAD TEST RESULTS FOR EAGLE LAKE GRAVEL MIXTURES

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus 0.5 N_f, kPa (psi)	Poisson's Ratio
10 (50)	215 (31.2)	3.5	2,215 (138.3)	10.9	45,236	5,668,500 (821,933)	-0.08
		4.0	2,223 (138.8)	9.9	78,765	7,903,300 (1,145,978)	-0.13
		4.0	2,211 (138.0)	10.4	44,833	8,789,210 (1,274,435)	-0.08
		4.5	2,350 (146.7)	4.1	93,545	6,758,630 (980,002)	-0.13
		4.5	2,355 (147.0)	3.9	63,938	7,059,600 (1,023,642)	-0.13
		4.5	2,339 (146.0)	4.6	88,108	11,264,120 (1,633,298)	0.01
		5.0	2,313 (144.4)	4.9	54,702	7,923,510 (1,148,909)	-0.06
		5.0	2,332 (145.6)	4.1	39,503	5,465,330 (792,473)	-0.14
		5.0	2,324 (145.1)	4.5	49,749	5,812,330 (842,788)	-0.03
24 (75)	40 (5.8)	3.5	2,343 (146.3)	5.8	13,072	2,550,260 (369,787)	0.52
		3.5	2,352 (146.8)	5.4	11,111	3,129,080 (453,717)	0.08
		3.5	2,329 (145.4)	6.4	7,636	3,202,380 (464,345)	0.34
		4.0	2,374 (148.2)	3.8	47,869	3,271,750 (474,404)	-0.08
		4.0	2,374 (146.5)	4.9	12,500	4,702,630 (681,882)	1.25
		4.0	2,352 (146.8)	4.7	12,540	3,147,940 (456,452)	0.11
		4.5	2,360 (147.3)	3.7	40,240	2,076,750 (301,129)	0.01
		4.5	2,355 (147.0)	3.9	40,671	2,526,520 (366,346)	0.00
		5.0	2,342 (146.2)	3.8	30,700	4,376,180 (634,546)	0.91
		5.0	2,348 (146.6)	3.5	15,920	8,699,380 (1,261,410)	2.13
	120 (17.4)	3.5	2,343 (146.3)	5.8	1,121	3,124,390 (453,037)	0.74
		3.5	2,331 (145.5)	6.3	869	3,354,880 (486,457)	0.84
		4.0	2,345 (146.4)	5.0	1,355	3,417,720 (495,569)	0.69
		4.0	2,366 (147.7)	4.1	2,109	2,443,100 (354,250)	0.07

(continued)

TABLE F-1. (Continued)

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus $0.5\ N_f$, kPa (psi)	Poisson's Ratio
24 (75)	120 (17.4)	4.5	2,362 (147.5)	3.6	2,308	3,438,570 (498,593)	0.80
		4.5	2,356 (147.1)	3.8	1,808	2,824,870 (409,606)	0.42
		5.0	2,334 (145.7)	4.1	2,044	3,349,700 (485,706)	0.97
		5.0	2,363 (147.5)	2.9	1,695	4,123,660 (597,931)	1.57
38 (100)	25 (3.6)	3.5	2,322 (145.0)	6.6	880	786,630 (114,061)	0.18
		4.0	2,371 (148.0)	3.9	1,784	901,810 (130,763)	0.38
		4.0	2,366 (147.7)	4.1	1,077	781,140 (113,266)	0.27
		4.5	2,368 (147.8)	3.4	2,560	871,500 (126,367)	0.38
		4.5	2,362 (147.5)	3.6	2,906	1,280,980 (185,742)	0.48
		5.0	2,350 (146.7)	3.4	1,736	697,340 (101,115)	0.35
		5.0	2,344 (146.3)	3.7	1,948	787,120 (114,132)	0.32
	40 (5.8)	3.5	2,340 (146.1)	5.9	361	1,289,460 (186,972)	0.77
		3.5	2,345 (146.4)	5.7	467	934,490 (135,501)	0.44
		4.0	2,371 (148.0)	3.9	1,366	1,058,970 (153,536)	0.30
		4.0	2,332 (145.6)	5.5	681	832,790 (120,754)	0.33
		4.5	2,368 (147.8)	3.4	1,055	1,263,770 (183,246)	0.55
		4.5	2,379 (148.5)	2.9	1,596	759,850 (110,178)	0.24
		5.0	2,340 (146.1)	3.8	820	681,720 (98,850)	0.25
		5.0	2,332 (145.6)	4.1	705	664,490 (96,351)	0.41

TABLE F-2. SPECIMEN PROPERTIES AND REPEATED-LOAD TEST RESULTS FOR LUBBOCK LIMESTONE MIXTURES

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus $0.5\ N_f$, kPa (psi)	Poisson's Ratio
		6.5	2,137 (133.4)	11.9	18,137	4,376,860 (634,645)	0.09
		7.0	2,191 (136.8)	9.0	25,982	5,048,340 (732,009)	0.10
		7.0	2,151 (134.3)	10.6	21,274	5,889,720 (854,009)	0.19
		7.5	2,190 (136.7)	8.4	35,355	4,415,300 (640,218)	0.01
	550 (79.8)	7.5	2,182 (136.2)	8.7	69,579	6,722,980 (974,832)	0.29
		7.5	2,188 (136.6)	8.4	56,587	7,560,810 (1,096,317)	0.28
		8.0	2,187 (136.5)	7.8	27,357	5,110,080 (740,962)	0.13
		8.0	2,161 (134.9)	8.9	30,679	5,647,840 (818,937)	0.25
10 (50)		6.5	2,175 (135.8)	10.3	1,508	6,437,030 (933,370)	0.28
		6.5	2,111 (131.8)	12.9	875	4,956,660 (718,715)	0.22
		7.0	2,182 (136.2)	9.4	2,084	5,952,360 (863,092)	0.11
		7.0	2,167 (135.3)	10.0	1,713	5,980,710 (867,203)	0.17
	1000 (145)	7.5	2,182 (136.2)	8.7	1,558	5,527,250 (801,451)	0.11
		7.5	2,178 (136.0)	8.8	2,441	4,896,770 (710,032)	0.10
		7.5	2,185 (136.4)	8.6	2,425	5,440,940 (788,936)	0.21
		8.0	2,172 (135.6)	8.4	2,392	6,333,100 (918,299)	0.22
		8.0	2,164 (135.1)	8.8	1,288	4,711,260 (683,133)	0.28
		6.5	2,153 (134.4)	11.2	21,088	2,351,750 (341,004)	0.07
		7.0	2,148 (134.1)	10.8	18,570	2,221,160 (322,069)	0.13
		7.0	2,161 (134.9)	10.2	27,150	2,624,620 (380,570)	0.07
		7.0	2,207 (137.8)	8.3	39,381	3,758,610 (544,999)	0.16
24 (75)	150 (21.7)	7.5	2,182 (136.2)	8.7	27,500	2,726,220 (395,302)	0.12
		7.5	2,196 (137.1)	8.1	40,000	2,305,690 (334,324)	0.01
		7.5	2,188 (136.6)	8.4	73,144	2,673,930 (387,720)	0.10
		8.0	2,180 (136.1)	8.1	24,663	2,897,870 (420,191)	0.03
		8.0	2,177 (135.9)	8.2	12,630	2,562,410 (371,549)	0.25

(continued)

TABLE F-2. (Continued)

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus 0.5 N_f, kPa (psi)	Poisson's Ratio
		6.5	2,135 (133.3)	11.9	2,768	2,094,890 (303,759)	0.09
		7.0	2,175 (135.8)	9.6	3,312	2,842,930 (412,225)	0.13
		7.0	2,169 (135.4)	9.9	4,382	2,973,750 (431,194)	0.21
		7.0	2,175 (135.8)	9.6	4,880	2,984,680 (432,779)	-0.08
24 (75)	250 (36.2)	7.5	2,178 (136.0)	8.8	3,070	3,338,830 (484,131)	0.33
		7.5	2,187 (136.5)	8.5	3,118	2,802,250 (406,326)	0.14
		7.5	2,198 (137.2)	8.0	4,881	2,351,910 (341,028)	0.13
		8.0	2,178 (136.0)	8.2	3,448	3,018,890 (437,739)	0.33
		8.0	2,185 (136.4)	7.9	2,066	2,610,440 (378,514)	0.09
		6.5	2,132 (133.1)	12.1	9,437	1,050,880 (152,377)	0.27
		7.0	2,174 (135.7)	9.7	6,430	1,274,600 (184,817)	0.37
		7.0	2,174 (135.7)	9.7	6,161	1,624,010 (235,481)	0.52
	80 (11.6)	7.5	2,204 (137.6)	7.8	12,249	1,462,630 (212,082)	0.46
		7.5	2,191 (136.8)	8.3	9,964	1,304,500 (189,152)	0.31
		8.0	2,182 (136.2)	8.0	6,209	1,103,700 (160,037)	0.51
		8.0	2,174 (135.7)	8.4	7,732	1,330,650 (192,944)	0.56
38 (100)		6.5	2,098 (131.0)	13.5	458	1,274,590 (184,815)	0.78
		7.0	2,187 (136.5)	9.2	1,321	1,594,940 (231,266)	0.67
		7.0	2,170 (135.5)	9.8	2,333	1,401,120 (203,163)	0.28
	120 (17.4)	7.5	2,177 (135.9)	8.9	2,109	1,242,140 (180,111)	0.59
		7.5	2,174 (135.7)	9.0	3,065	1,826,880 (264,898)	0.56
		8.0	2,175 (135.8)	8.3	1,854	1,176,450 (170,586)	0.62
		8.0	2,187 (136.5)	7.8	1,036	1,399,090 (202,868)	0.47

TABLE F-3. SPECIMEN PROPERTIES AND REPEATED-LOAD TEST RESULTS FOR LUFKIN SAND MIXTURES

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus $0.5\ N_f$, kPa (psi)	Poisson's Ratio
10 (50)	280 (40.6)	5.0	2,228 (139.1)	8.8	27,500	5,551,400 (804,953)	0.13
		5.5	2,225 (138.9)	8.2	14,898	2,074,410 (300,790)	0.23
		5.5	2,198 (137.2)	9.4	26,170	3,557,750 (515,874)	0.25
		6.0	2,270 (141.7)	5.7	22,862	2,547,870 (369,441)	0.15
		6.0	2,249 (140.4)	6.6	25,521	3,000,690 (435,100)	0.32
		6.5	2,255 (140.8)	5.6	32,156	2,882,260 (417,928)	0.26
		6.5	2,263 (141.3)	5.3	20,919	2,668,380 (386,915)	0.16
		7.0	2,257 (140.9)	4.9	12,198	2,807,530 (407,092)	0.29
	600 (87.0)	5.0	2,204 (137.6)	9.8	872	3,245,990 (470,669)	0.24
		5.5	2,207 (137.8)	9.0	848	3,967,630 (575,307)	0.36
		5.5	2,247 (140.3)	7.3	1,806	3,903,230 (565,968)	0.31
		6.0	2,268 (141.6)	5.8	1,282	2,888,790 (418,875)	0.21
		6.0	2,260 (141.1)	6.1	1,477	3,904,270 (566,119)	0.28
		6.5	2,254 (140.7)	5.7	1,520	3,341,050 (484,452)	0.17
		6.5	2,243 (140.0)	6.2	992	3,508,790 (508,774)	0.37
		7.0	2,263 (141.3)	4.7	1,074	3,245,610 (470,614)	0.23
24 (75)	60 (8.7)	5.0	2,270 (141.7)	7.1	31,484	1,002,010 (145,291)	0.14
		5.0	2,220 (138.6)	9.1	21,210	1,102,520 (159,866)	0.33
		5.5	2,254 (140.7)	7.1	25,518	999,470 (144,923)	0.32
		5.5	2,230 (139.2)	8.1	25,619	1,074,480 (155,800)	0.45
		6.0	2,270 (141.7)	5.7	26,817	1,263,130 (183,154)	0.75
		6.0	2,252 (140.6)	6.4	26,700	1,259,580 (182,639)	0.65
		6.5	2,276 (142.1)	4.8	33,291	978,700 (141,911)	0.28
		6.5	2,267 (141.5)	5.2	17,076	914,240 (132,565)	0.44

(continued)

TABLE F-3. (Continued)

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus $0.5\ N_f$, kPa (psi)	Poisson's Ratio
		5.0	2,249 (140.4)	7.9	968	984,260 (142,718)	0.30
		5.0	2,206 (137.7)	9.7	933	1,112,850 (161,364)	0.36
		5.5	2,252 (140.6)	7.1	1,209	1,130,500 (163,922)	0.37
24 (75)	180 (26.1)	5.5	2,235 (139.5)	7.9	1,026	1,150,540 (166,829)	0.37
		6.0	2,249 (140.4)	6.6	722	1,168,160 (169,383)	0.55
		6.0	2,287 (142.8)	5.0	1,164	1,279,660 (185,551)	0.34
		6.5	2,281 (142.4)	4.6	848	1,211,890 (175,724)	0.56
		6.5	2,271 (141.8)	5.0	664	996,490 (144,491)	0.42
		5.0	2,239 (139.8)	8.3	13,074	410,450 (59,516)	0.49
		5.5	2,247 (140.3)	7.3	12,533	473,250 (68,622)	0.47
		5.5	2,243 (140.0)	7.5	21,984	446,680 (64,768)	0.44
		6.0	2,262 (141.2)	6.0	21,542	455,300 (66,018)	0.76
		6.0	2,307 (144.0)	4.2	79,635	413,740 (59,993)	0.62
		6.0	2,292 (143.1)	4.8	22,648	410,690 (59,550)	0.49
38 (100)	20 (2.9)	6.5	2,287 (142.8)	4.3	53,000	457,260 (66,303)	0.65
		6.5	2,273 (141.9)	4.9	36,232	366,030 (53,075)	0.53
		6.5	2,299 (143.5)	3.8	36,650	308,640 (44,753)	0.35
		7.0	2,292 (143.1)	3.4	46,047	334,600 (48,517)	0.66
		7.0	2,292 (143.1)	3.4	65,988	228,450 (33,126)	0.44
		7.0	2,247 (140.3)	5.3	25,916	463,280 (67,176)	0.45
		7.0	2,238 (139.7)	5.7	27,325	483,130 (70,054)	0.56
		7.5	2,275 (142.0)	3.5	11,560	253,160 (36,708)	0.49
		8.0	2,263 (141.3)	3.3	3,930	290,190 (42,078)	0.77

(continued)

TABLE F-3. (Continued)

Temperature, °C (°F)	Stress Level, kPa (psi)	Asphalt Content, %	Density, kg/m^3 (pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus $0.5\ N_f$, kPa (psi)	Poisson's Ratio
		5.0	2,223 (138.8)	9.0	1,711	302,500 (43,862)	0.72
		5.5	2,234 (139.5)	7.9	1,846	417,410 (60,524)	0.53
		5.5	2,259 (141.0)	6.9	2,469	491,240 (71,230)	0.39
		6.0	2,257 (140.9)	6.2	1,628	450,030 (65,254)	0.71
		6.0	2,297 (143.4)	4.6	4,790	450,570 (65,333)	0.56
		6.0	2,273 (141.9)	5.6	1,824	379,410 (55,014)	0.74
38 (100)	40 (5.8)	6.5	2,289 (142.9)	4.2	4,749	423,160 (61,358)	0.57
		6.5	2,276 (142.1)	4.8	4,376	332,000 (48,140)	0.47
		6.5	2,297 (143.4)	3.9	3,375	335,160 (48,599)	0.42
		7.0	2,286 (142.7)	3.7	2,795	336,390 (48,776)	0.42
		7.0	2,281 (142.4)	3.9	2,665	443,390 (64,291)	1.06
		7.5	2,275 (142.0)	3.5	1,250	300,290 (43,542)	0.66
		8.0	2,263 (141.2)	3.3	830	268,230 (38,894)	0.45

APPENDIX G

PERMANENT DEFORMATION CHARACTERISTICS

TABLE G-1. PERMANENT DEFORMATION CHARACTERISTICS FOR EAGLE LAKE GRAVEL MIXTURES

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation mm/cycle	Permanent Deformation in./cycle
		3.5	45,236	5.84×10^{-5}	2.30×10^{-6}
		4.0	78,765	5.21×10^{-5}	2.05×10^{-6}
		4.5	93,545	4.72×10^{-5}	1.86×10^{-6}
10 (50)	215 (31.2)	4.5	63,938	3.81×10^{-5}	1.50×10^{-6}
		4.5	88,108	5.48×10^{-5}	2.16×10^{-6}
		5.0	54,702	2.01×10^{-4}	7.92×10^{-6}
		5.0	39,503	1.89×10^{-4}	7.44×10^{-6}
		5.0	49,749	1.57×10^{-4}	6.20×10^{-6}
		3.5	13,072	2.28×10^{-4}	8.96×10^{-6}
		3.5	7,636	3.86×10^{-4}	1.52×10^{-5}
		3.5	11,111	2.84×10^{-4}	9.76×10^{-6}
		4.0	47,869	2.59×10^{-4}	1.02×10^{-5}
		4.0	12,500	2.57×10^{-4}	1.01×10^{-5}
	40 (5.8)	4.0	12,540	6.65×10^{-5}	2.62×10^{-6}
		4.5	81,979	7.37×10^{-5}	2.90×10^{-6}
		4.5	40,240	1.25×10^{-4}	4.92×10^{-6}
		4.5	40,670	1.46×10^{-4}	5.76×10^{-6}
		5.0	15,920	6.30×10^{-4}	2.48×10^{-5}
24 (75)					
		3.5	1,121	3.00×10^{-3}	1.18×10^{-4}
		3.5	869	5.59×10^{-3}	2.20×10^{-4}
		4.0	1,355	3.76×10^{-3}	1.48×10^{-4}
	120 (17.4)	4.0	2,109	2.77×10^{-3}	1.09×10^{-4}
		4.5	2,308	3.35×10^{-3}	1.32×10^{-4}
		4.5	1,808	4.78×10^{-3}	1.88×10^{-4}
		5.0	1,695	5.56×10^{-3}	2.19×10^{-4}

(continued)

TABLE G-1. (continued)

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation mm/cycle	Permanent Deformation in./cycle
38 (100)	25 (3.6)	3.5	880	4.65×10^{-3}	1.83×10^{-4}
		4.0	1,784	1.78×10^{-3}	7.00×10^{-5}
		4.0	1,077	3.43×10^{-3}	1.35×10^{-4}
		4.5	2,560	2.22×10^{-3}	8.76×10^{-5}
		4.5	2,906	1.99×10^{-3}	7.84×10^{-5}
		5.0	1,736	5.18×10^{-3}	2.04×10^{-4}
		5.0	1,948	2.59×10^{-3}	1.02×10^{-4}
	40 (5.9)	3.5	361	3.12×10^{-3}	1.23×10^{-4}
		3.5	467	8.58×10^{-3}	3.38×10^{-4}
		4.0	1,366	8.36×10^{-3}	3.29×10^{-4}
		4.0	681	6.60×10^{-3}	2.60×10^{-4}
		4.5	1,055	7.06×10^{-3}	2.78×10^{-4}
		4.5	1,596	5.56×10^{-3}	2.19×10^{-4}
		5.0	820	3.05×10^{-3}	1.20×10^{-4}
		5.0	705	1.54×10^{-2}	6.08×10^{-4}

TABLE G-2. PERMANENT DEFORMATION CHARACTERISTICS FOR LUBBOCK LIMESTONE MIXTURES

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation mm/cycle	Permanent Deformation in./cycle
10 (50)	550 (79.8)	6.5	18,137	8.18×10^{-5}	3.22×10^{-6}
		7.0	25,982	5.79×10^{-5}	2.28×10^{-6}
		7.0	21,274	8.13×10^{-5}	3.20×10^{-6}
		7.5	35,355	6.07×10^{-5}	2.39×10^{-6}
		7.5	69,579	3.83×10^{-5}	1.51×10^{-6}
		7.5	56,587	2.32×10^{-5}	9.12×10^{-7}
		8.0	27,357	1.26×10^{-4}	4.96×10^{-6}
		8.0	30,679	2.44×10^{-4}	9.60×10^{-6}
	1000 (14.5)	6.5	675	2.41×10^{-3}	9.48×10^{-5}
		7.0	2,084	8.08×10^{-4}	3.18×10^{-5}
		7.0	1,713	1.26×10^{-3}	4.96×10^{-5}
		7.5	1,558	1.57×10^{-3}	6.20×10^{-5}
		7.5	2,441	1.15×10^{-3}	4.52×10^{-5}
		7.5	2,425	9.58×10^{-4}	3.77×10^{-5}
		8.0	2,392	1.23×10^{-3}	4.84×10^{-5}
		8.0	1,288	4.78×10^{-3}	1.88×10^{-4}
24 (75)	150 (21.7)	6.5	21,088	4.70×10^{-6}	1.85×10^{-6}
		7.0	18,570	6.73×10^{-5}	2.65×10^{-6}
		7.0	27,150	5.16×10^{-5}	2.03×10^{-6}
		7.0	39,381	1.45×10^{-4}	5.72×10^{-6}
		7.5	27,500	2.72×10^{-4}	1.07×10^{-5}
		7.5	40,000	4.78×10^{-5}	1.88×10^{-6}
		7.5	73,144	3.22×10^{-5}	1.27×10^{-6}
		8.0	24,663	3.10×10^{-4}	1.22×10^{-5}
		8.0	12,630	8.71×10^{-4}	3.43×10^{-5}

(continued)

TABLE G-2. (continued)

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation, mm/cycle	Permanent Deformation, in./cycle
		6.5	2,768	6.15×10^{-4}	2.42×10^{-5}
		7.0	3,312	5.89×10^{-4}	2.32×10^{-5}
		7.0	4,382	6.25×10^{-4}	2.46×10^{-5}
		7.0	4,880	1.20×10^{-3}	4.72×10^{-5}
24 (75)	250 (36.2)	7.5	3,070	2.67×10^{-3}	1.05×10^{-4}
		7.5	3,118	1.13×10^{-3}	4.44×10^{-5}
		7.5	4,881	9.45×10^{-4}	3.72×10^{-5}
		8.0	3,448	1.96×10^{-3}	7.72×10^{-5}
		8.0	2,066	4.77×10^{-3}	1.88×10^{-4}
		6.5	9,437	1.40×10^{-3}	5.52×10^{-5}
		7.0	6,430	5.16×10^{-4}	2.03×10^{-5}
		7.0	6,161	2.77×10^{-4}	1.09×10^{-5}
	80 (11.6)	7.5	12,249	6.10×10^{-4}	2.40×10^{-5}
		7.5	9,964	3.05×10^{-4}	1.20×10^{-5}
		8.0	6,209	1.87×10^{-3}	7.36×10^{-5}
		8.0	7,732	8.10×10^{-4}	3.19×10^{-5}
38 (100)					
		6.5	458	3.56×10^{-3}	1.40×10^{-4}
		7.0	1,321	3.71×10^{-3}	1.46×10^{-4}
		7.0	2,333	1.09×10^{-3}	4.28×10^{-5}
	120 (17.4)	7.5	2,109	3.89×10^{-3}	1.53×10^{-4}
		7.5	3,065	1.13×10^{-3}	4.44×10^{-5}
		8.0	1,854	5.03×10^{-3}	1.98×10^{-4}
		8.0	1,036	6.45×10^{-3}	2.54×10^{-4}

TABLE G-3. PERMANENT DEFORMATION CHARACTERISTICS FOR LUFKIN SAND MIXTURES

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation, mm/cycle	Permanent Deformation, in./cycle
10 (50)	280 (40.6)	5.0	27,500	2.34×10^{-4}	9.20×10^{-6}
		5.5	14,898	4.55×10^{-4}	1.79×10^{-5}
		5.5	26,170	2.41×10^{-4}	9.48×10^{-6}
		6.0	22,862	2.87×10^{-4}	1.13×10^{-5}
		6.0	25,521	2.35×10^{-4}	9.24×10^{-6}
		6.5	32,156	2.05×10^{-4}	8.08×10^{-6}
		6.5	20,919	3.53×10^{-4}	1.39×10^{-5}
		7.0	12,198	5.69×10^{-4}	2.24×10^{-5}
	600 (87.0)	5.0	872	5.97×10^{-3}	2.35×10^{-4}
		5.5	848	7.14×10^{-3}	2.81×10^{-4}
		5.5	1,806	3.25×10^{-3}	1.28×10^{-4}
		6.0	1,282	4.75×10^{-3}	1.87×10^{-4}
		6.0	1,477	4.09×10^{-3}	1.61×10^{-4}
		6.5	1,520	4.14×10^{-3}	1.63×10^{-4}
		6.5	992	6.63×10^{-3}	2.61×10^{-4}
		7.0	1,074	7.82×10^{-3}	3.08×10^{-4}
24 (75)	60 (8.7)	5.0	21,210	2.46×10^{-4}	9.68×10^{-6}
		5.0	31,484	1.41×10^{-4}	5.56×10^{-6}
		5.5	25,518	2.89×10^{-4}	1.14×10^{-5}
		5.5	25,619	2.23×10^{-4}	8.80×10^{-6}
		6.0	26,817	2.38×10^{-4}	9.36×10^{-6}
		6.0	26,700	2.41×10^{-4}	9.50×10^{-6}
		6.5	33,291	5.49×10^{-4}	2.16×10^{-5}
		6.5	17,076	5.38×10^{-4}	2.12×10^{-5}

(continued)

TABLE G-3. (continued)

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation, mm/cycle	Permanent Deformation, in./cycle
24 (75)	180 (26.1)	5.0	933	8.00×10^{-3}	3.15×10^{-4}
		5.0	968	6.88×10^{-3}	2.71×10^{-4}
		5.5	1,209	7.24×10^{-3}	2.85×10^{-4}
		5.5	1,026	7.62×10^{-3}	3.00×10^{-4}
		6.0	722	1.18×10^{-2}	4.64×10^{-4}
		6.0	1,164	7.72×10^{-3}	3.04×10^{-4}
		6.5	848	1.22×10^{-2}	4.80×10^{-4}
		6.5	664	1.61×10^{-2}	6.36×10^{-4}
38 (100)	20 (2.9)	5.0	13,074	2.87×10^{-4}	1.13×10^{-5}
		5.5	12,533	2.72×10^{-4}	1.07×10^{-5}
		5.5	21,984	1.61×10^{-4}	6.36×10^{-6}
		6.0	21,542	1.55×10^{-4}	6.12×10^{-6}
		6.0	22,648	3.91×10^{-5}	1.54×10^{-6}
		6.0	79,635	1.31×10^{-4}	5.16×10^{-6}
		6.5	53,000	7.26×10^{-5}	2.86×10^{-6}
		6.5	36,232	1.31×10^{-4}	5.16×10^{-6}
		6.5	36,650	1.31×10^{-4}	5.16×10^{-6}
		7.0	46,047	1.40×10^{-4}	5.52×10^{-6}
		7.0	65,988	9.65×10^{-5}	3.80×10^{-6}
		7.0	25,916	1.28×10^{-4}	5.04×10^{-6}
		7.0	27,325	1.75×10^{-4}	6.88×10^{-6}
		7.5	11,560	6.73×10^{-4}	2.65×10^{-5}
		8.0	3,930	2.67×10^{-3}	1.05×10^{-4}

(continued)

TABLE G-3. (continued)

Temperature, °C (°F)	Applied Stress, kPa (psi)	Asphalt Content, %	Fatigue Life, Cycles	Permanent Deformation, mm/cycle	Permanent Deformation, in./cycle
		5.0	1,711	3.51×10^{-3}	1.38×10^{-4}
		5.5	1,846	3.48×10^{-3}	1.37×10^{-4}
		5.5	2,469	2.27×10^{-3}	8.92×10^{-5}
		6.0	1,628	4.04×10^{-3}	1.59×10^{-4}
		6.0	1,824	1.40×10^{-3}	5.52×10^{-5}
		6.0	4,790	4.34×10^{-3}	1.71×10^{-4}
38 (100)	40 (5.8)	6.5	4,749	1.54×10^{-3}	6.08×10^{-5}
		6.5	4,376	1.84×10^{-3}	7.24×10^{-5}
		6.5	3,375	2.64×10^{-3}	1.04×10^{-4}
		7.0	2,665	3.48×10^{-3}	1.37×10^{-4}
		7.0	2,795	3.40×10^{-3}	1.34×10^{-4}
		7.5	1,250	1.26×10^{-2}	4.96×10^{-4}
		8.0	830	1.61×10^{-2}	6.36×10^{-4}