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16. Abstract

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Two aggregates, a rounded gravel and a crushed limestone, each with various soil binder contents, were mixed with a range of asphalt contents to produce test specimens. The engineering properties were compared for the various soil binder contents. Results of these comparisons indicated that the various engineering properties could be maximized at relatively low soil binder contents and at lower asphalt contents.

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THE EFFECTS OF SOIL BINDER AND MOISTURE ON BLACKBASE MIXTURES

by

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Thomas W. Kennedy

Research Report Number 183-12

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

May 1979

PREFACE

This is the twelfth 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 effect of soil binder content on the behavior of blackbase mixtures used in Texas. The evaluation was based upon the results obtained using the static and repeated-load indirect tensile tests on two blackbase mixtures which have been used in Texas.

This report was completed with the assistance of many people. Special appreciation is due James N. Anagnos and Pat Hardeman for their assistance with 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 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 acknowledged.

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Thomas W. Kennedy

May 1979

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. 183-7, "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.

Report No. 183-12, "The Effects of Soil Binder and Moisture on Blackbase Mixtures," by Wei-Chou V. Ping and Thomas W. Kennedy, summarizes the results of a study to evaluate the effect of soil binder content on the engineering properties of blackbase paving mixtures.

ABSTRACT

This report describes a study which was undertaken to evaluate the effect of the amount of soil binder on the engineering properties of asphalt-treated paving materials. For this study soil binder was considered to be aggregate finer than U. S. standard sieve size No. 40. The static and repeated-load indirect tensile tests were used to measure engineering properties of asphalt mixtures for purposes of mixture design and evaluation.

Two aggregates, a rounded gravel and a crushed limestone, each with various soil binder contents, were mixed with a range of asphalt contents to produce test specimens. The engineering properties were compared for the various soil binder contents. Results of these comparisons indicated that the various engineering properties could be maximized at relatively low soil binder contents and at lower asphalt contents.

KEY WORDS: blackbase, asphalt concrete, asphalt-treated, asphalt stabilized, soil binder content, mixture design, indirect tensile test, engineering properties, tensile strength, modulus of elasticity, resilient modulus, fatigue life, permanent deformation.

SUMMARY

The purpose of this study was to evaluate the effects of soil binder content on the behavior of blackbase mixtures used in Texas. For this study soil binder was considered to be aggregate finer than U. S. standard sieve size No. 40. The evaluation was based upon a comparison and analysis of engineering properties obtained using the static and repeated-load indirect tensile tests on mixtures with various soil binder contents.

For this study two blackbase mixtures, a rounded gravel and field sand and a crushed limestone. Each of these aggregates has been used in a black-base mixtures. Various engineering properties were evaluated at various soil binder contents and asphalt contents. The engineering properties evaluated were tensile strength, static modulus of elasticity, fatigue life, resilient modulus of elasticity, and resistance to permanent deformation. All tests were performed at 24°C (75°F). Most of the tests were conducted on specimens which were air dried; however, a limited number of tests were conducted on pressure wetted specimens to evaluate the influence of moisture content.

Generally, the results indicate that the various engineering properties were maximized at relatively low soil binder contents and at correspondingly lower asphalt contents. The optimum asphalt contents tended to decrease as the soil binder content decreased. The optimum soil binder contents for the various engineering properties ranged from 5 to 10 percent. In addition, the lowest optimum asphalt contents occurred at soil binder contents of 5 and 10 percent.

IMPLEMENTATION STATEMENT

Based on the findings of this study, it is recommended that additional consideration be given to the effects of soil binder content. The results of a limited amount of testing indicated that relative low binder contents maximize various engineering properties and minimize the optimum asphalt contents. Both effects suggest that lower binder contents are desirable; however, additional study is needed before final recommendations are made.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

The primary objective of this investigation was to evaluate the effect of soil binder content on the behavior and design of blackbase paving mixtures used in Texas. Soil binder is material which will pass the U. S. standard No. 40 sieve as defined by the Texas State Department of Highways and Public Transportation.

In practice, the acceptability of an aggregate gradation for use in asphalt mixtures is usually judged by its conformity to specified particular size limits (Ref 9). These limits have generally been established either on the basis of satisfactory experience with materials which meet selected gradation specifications or in terms of selected gradation patterns of natural or crushed material that are readily available. Thus, it is possible to have gradation limits which vary significantly but which will still produce satisfactory asphalt mixtures (Ref 14).

In Texas, a range of binder contents is specified as a part of the gradation requirements (Ref 27). However, the Texas Department of Highways and Public Transportation raised the question as to the effect of binder content and whether improved, or less costly, mixtures can be produced by specifying a limited binder content or by eliminating all specification requirements concerning binder content. To answer these questions, the Department of Highways and Transportation requested that a limited study be conducted to determine the effect of soil binder content on asphalt paving mixtures.

Previous investigations in Research Study 3-9-72-183, "Tensile Characterization of Highway Pavement Materials," successfully utilized the static and the repeated-load indirect tensile tests to measure engineering properties of asphalt mixtures for purposes of mixture design and evaluation. These tests were used, therefore, in this study to measure properties related to the distress modes of thermal or shrinkage cracking, fatigue cracking, and rutting. The experimental program is described in Chapter 2. Test results and findings are presented and discussed in Chapter 3, and the conclusions and recommendations are contained in Chapter 4.

CHAPTER 2. EXPERIMENTAL PROGRAM

The purpose of this investigation was to evaluate the effect of the amount of soil binder on the engineering properties of asphalt-treated paving materials. For this study, soil binder was considered to be aggregate finer than U. S. standard sieve size No. 40.

The basic experimental approach was to compare the engineering properties of asphalt mixtures composed of two representative types of aggregate, each with various soil binder contents. The two aggregates were a rounded river gravel and a crushed limestone (caliche), both of which are commonly used in pavement construction in Texas. By changing the quantity of soil binder, each selected aggregate gradation was mixed to produce laboratory specimens with asphalt contents in the range generally used for design.

This chapter describes the materials, aggregate gradations, equipment, and procedures used in the investigations.

MATERIALS

The two aggregates used in this investigation were obtained from Eagle Lake and Lubbock, Texas. Each of these aggregates has performed satisfactorily in pavements and has been studied in a previous investigation (Ref 24).

Eagle Lake Material

The Eagle Lake material was a mixture of four different aggregates; Lone Star coarse aggregate, Lone Star Gem sand, Tanner Walker sand, and Stiles coarse sand.

The 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 combined aggregates can be generally described as smooth-surface, angular, non-porous, crushed river gravel. This combination of aggregates was used in the blackbase construction of SH 71 south of Columbus, Texas.

The asphalt cement was an AC-20, produced at the Exxon refinery in Baytown, Texas and supplied by District 13 of the State Department of Highways and Public Transportation (DHT). The asphalt properties, as determined by the DHT, are summarized in Table 1.

Lubbock Material

The Lubbock material was a rough, sub-angular, porous, caliche type crushed limestone obtained from Long Pit, located approximately ten miles southeast of Lubbock, Texas. This aggregate was used for the blackbase construction of I-27 between the north loop of Lubbock and New Deal, Texas. 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.

AGGREGATE GRADATIONS

The gradation of the Eagle Lake gravel used for the construction of SH 71 and the gradation of the Lubbock limestone used for the construction of I-27 were used as basic gradations, one for each material.

The Eagle Lake field gradation was a mixture of the four different aggregates 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 field gradation contained 30 percent soil binder. For Eagle Lake gravel, the soil binder contents selected for study were 30, 20, 10, 5, and 0 percent. Gradations of the resulting mixtures are shown in Fig 1 and are listed in Table 2. The detailed individual aggregate gradations are contained in Appendix A.

For Lubbock limestone, the field gradation had 25 percent soil binder. The soil binder contents selected for study were 25, 10, 5, and 0 percent.

TABLE 1. SUMMARY OF ASPHALT CEMENT PROPERTIES*

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

*As reported by the State Department of Highways and Public Transportation

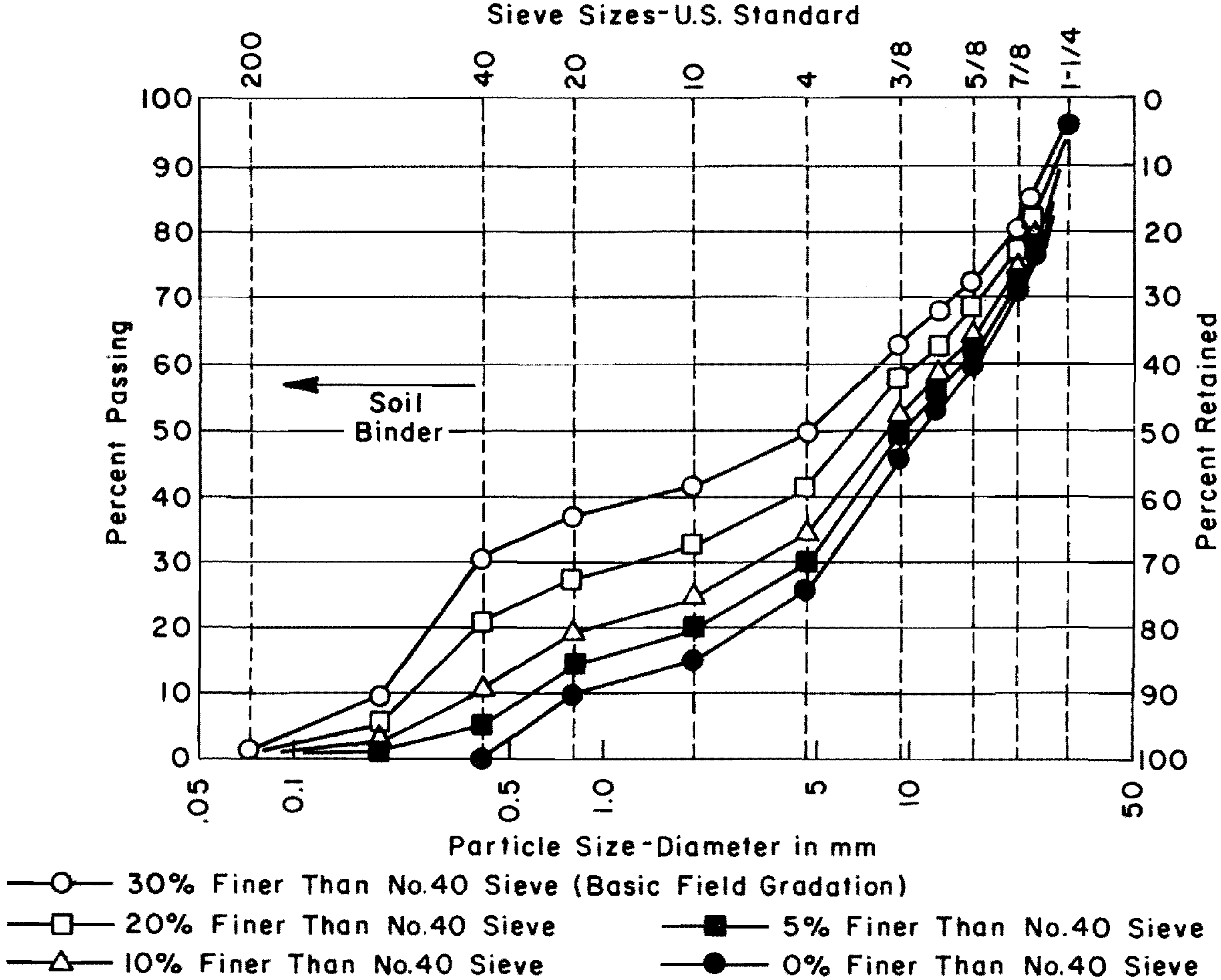


Fig 1. Gradations of Eagle Lake gravel mixtures.

TABLE 2. GRADATIONS OF MIXTURES

Material Description	% of Soil Binder	U.S. Standard Sieve Size, Cumulative % Retained											
		1 1/4"	1"	7/8"	5/8"	1/2"	3/8"	#4	#10	#20	#40	#80	#200
Eagle Lake gravel	30	3.4	15.0	19.2	27.3	32.4	37.1	51.4	58.9	63.0	69.6	91.1	99.2
	20	3.9	17.2	22.1	31.4	37.2	42.6	59.1	67.7	72.4	69.9	94.1	99.4
	10	4.4	19.4	24.8	35.2	41.8	47.8	66.4	76.2	81.5	90.0	97.1	99.8
	5	4.6	20.5	26.3	37.3	44.2	50.6	70.2	80.4	86.0	95.0	98.5	99.9
	0	4.9	21.6	27.6	39.2	46.6	53.3	73.9	84.6	90.5	100.0	100.0	100.0
Lubbock limestone	25			12.6	27.0	35.3	45.9	60.0	68.4		75.4		95.9
	10			15.0	32.2	42.1	54.8	71.6	81.6		90.0		98.3
	5			15.8	34.0	44.4	57.8	75.6	86.1		95.0		99.2
	0			16.7	35.8	46.8	60.9	79.6	90.7		100.0		100.0

Gradations of the resulting mixtures are shown in Fig 2 and are listed in Table 2.

The various gradations and percent soil binders were obtained by adding or removing material finer than the No. 40 sieve while maintaining a constant amount of the coarser material.

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 mixing (Ref 17). The complete mixing and compaction procedures are summarized below.

The aggregates were batched by dry weight, mixed with asphalt cement 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 specimen after gyration. This stress was maintained until the vertical deformation rate was less than 0.005 in. per 5-minute period, at which time the in-mold density, or AVR (Asphalt-Voids Ratio) density, was determined. The resulting specimen was approximately 152 mm (6 in.) in diameter and 200 mm (8 in.) high.

All specimens were allowed to cool in the compaction mold for about one hour before extrusion to prevent slumping of the specimen. This was necessary because a uniform diameter is desirable so that the loading strips used for indirect tensile testing will be in complete contact with the specimen. After extrusion from the mold, the specimens were allowed to cure overnight at room temperature. Smaller test specimens for the indirect tensile test were then cut from the top and bottom portions of the compacted specimen. The densities of these test specimens were calculated from their weights and physical dimensions.

The top and bottom specimens were cured overnight at a room temperature of approximately 24°C (75°F). Thus, the total curing time was two days. The sawed indirect tensile test specimens were generally 152 mm (6 in.) in diameter and about 84 mm (3.3 in.) in height.

To evaluate the effects of moisture, the exposed sawed faces of the test specimens were coated with a thin film of the same asphalt cement used in the mixture. Then the specimens were subjected to pressure wetting (Test Method

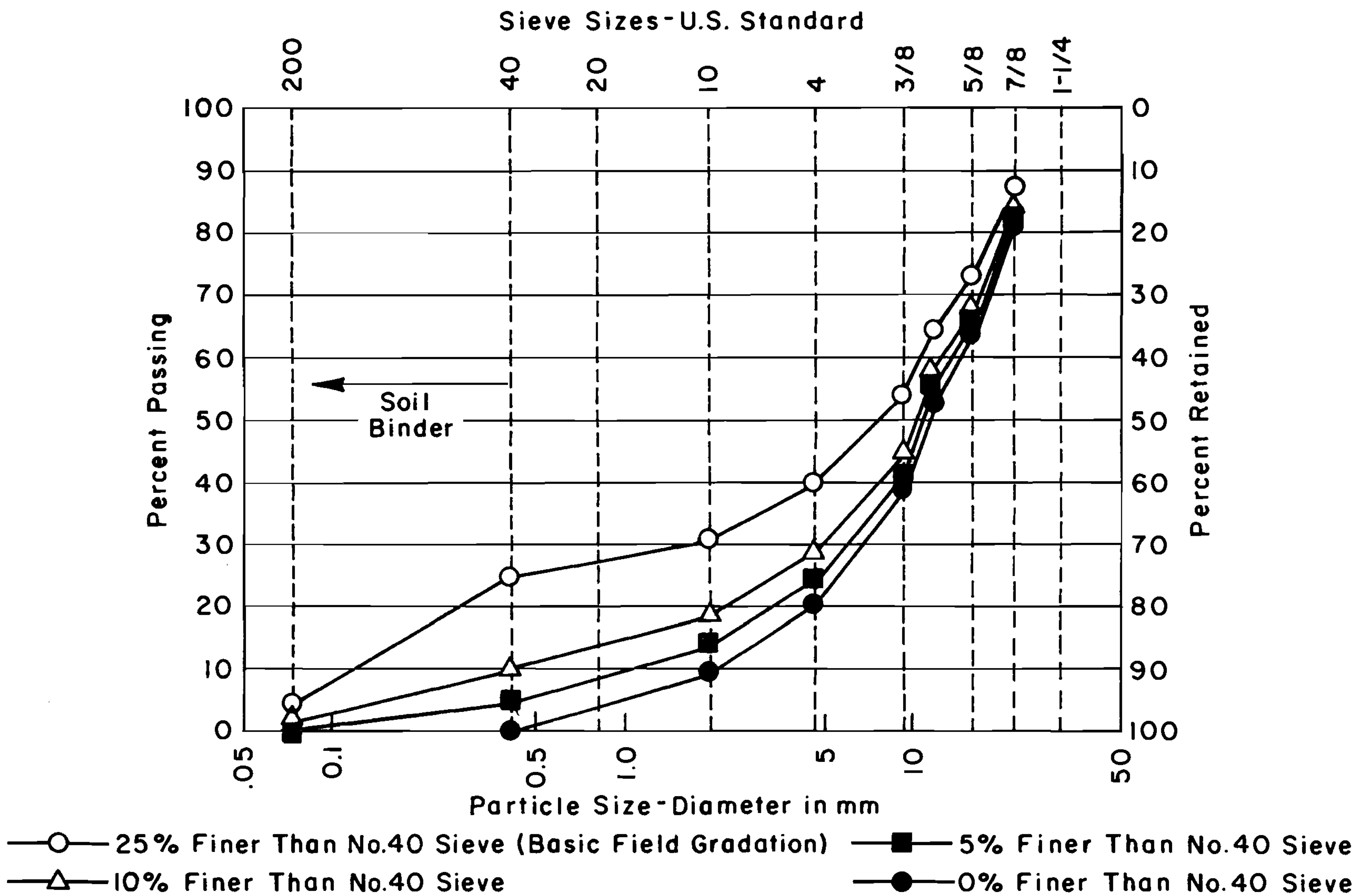


Fig 2. Gradations of Lubbock limestone mixtures.

Tex-109-E, Part IV). This procedure subjects a specimen to an 8274 kPa (1200 psi) hydrostatic water pressure at a water temperature of 65°C (150°F) for 15 minutes prior to actual testing using the indirect tensile test. The detailed procedure is described in Appendix B.

TESTING EQUIPMENT AND PROCEDURES

The testing equipment for the static and repeated-load indirect tensile tests was the same as that used in previous studies conducted at the Center for Highway Research. The basic testing apparatus was an MTS closed-loop electro-hydraulic loading system.

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

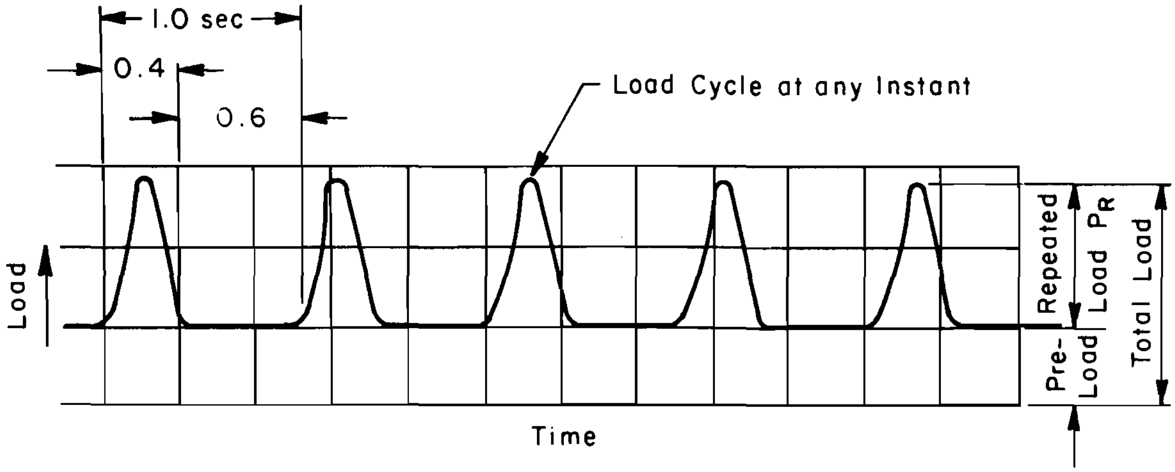
Vertical deformations were measured by a DC linear variable differential transducer (LVDT). Horizontal deformations were measured by two cantilevered arms with attached strain gauges. Both the load-vertical deformation and load-horizontal deformation relationships were recorded by a pair of X-Y plotters, Hewlett Packard Models 7001A and 7000AR for the repeated-load tests. A preload of 90 N (20 lb) was also applied. 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. Both the horizontal and vertical deformations were measured by DC-LVDT's and were recorded on the X-Y plotters. A typical load pulse and the resulting deformation relationships are shown in Figs 3 and 4. All tests were conducted at 24°C (75°F).

PROPERTIES

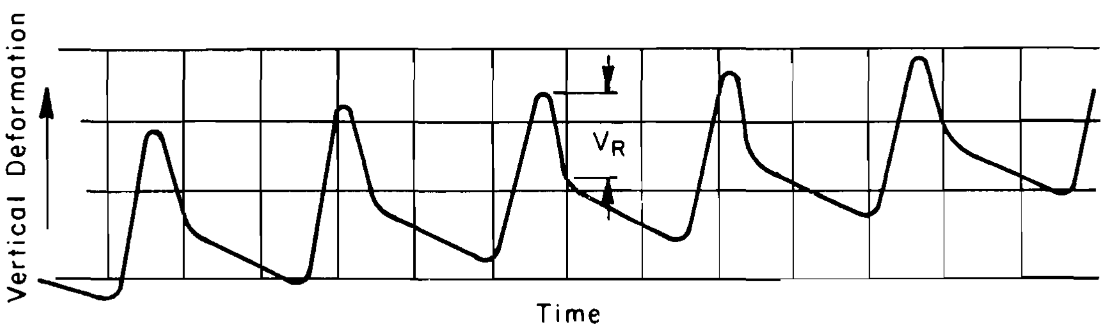
Several of the properties analyzed are related to the relevant pavement distress modes of (1) thermal or shrinkage cracking, (2) fatigue cracking, and (3) permanent deformations, or rutting.

The properties analyzed were

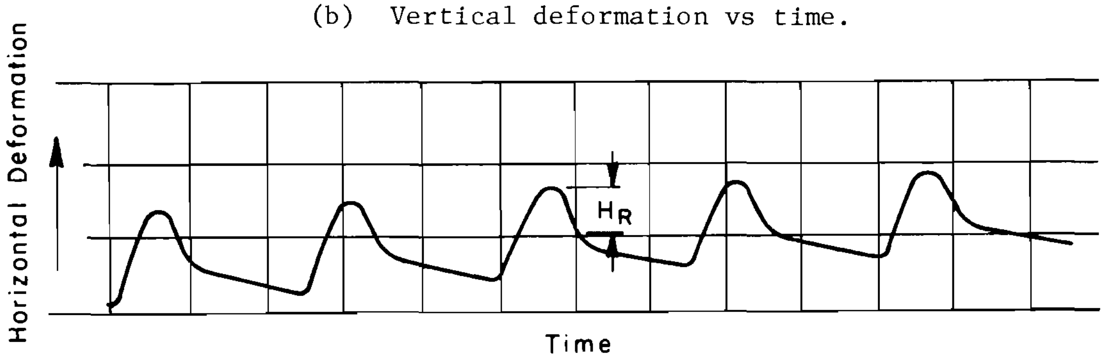
AVR density,
total air voids,
tensile strength,



(a) Load-time pulse.



(b) Vertical deformation vs time.



(c) Horizontal deformation vs time.

Fig 3. Typical load pulse and relationships between deformations and time for repeated-load indirect tensile test.

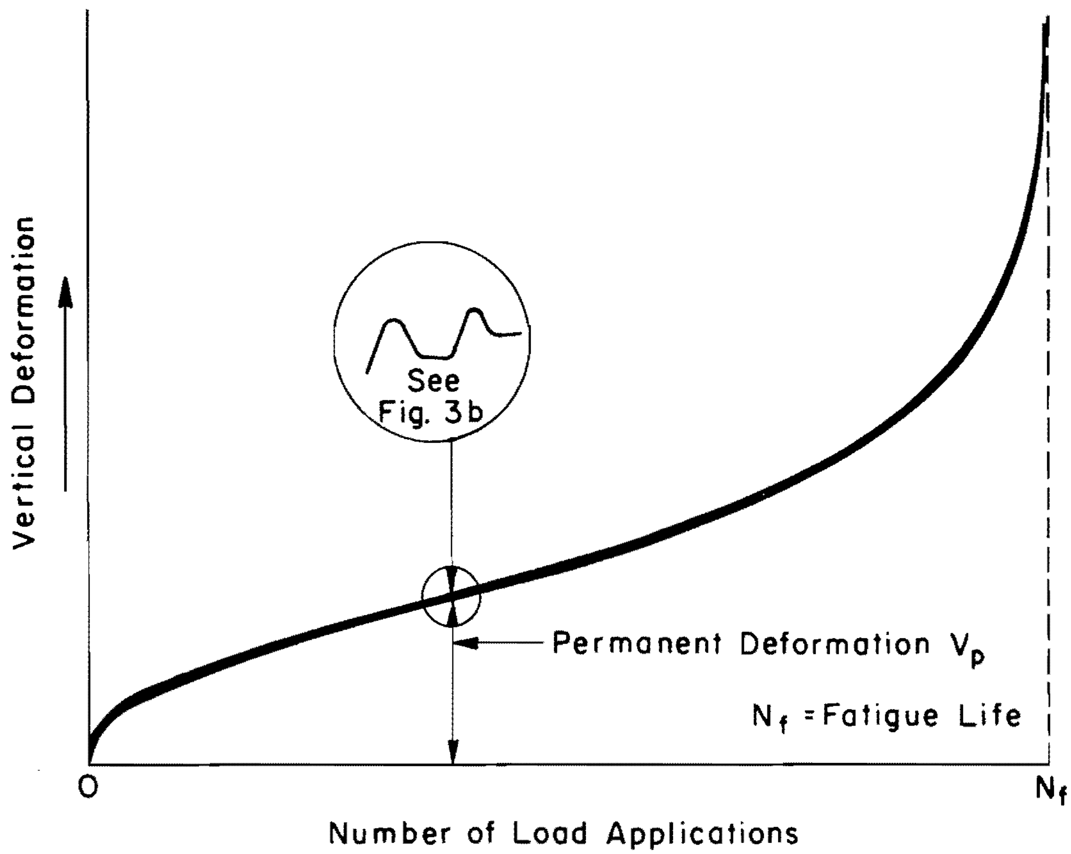


Fig 4. Relationship between number of load applications and vertical deformations for the repeated-load indirect tensile test.

static Poisson's ratio,
static modulus of elasticity,
fatigue life,
resilient Poisson's ratio,
resilient modulus of elasticity, and
permanent deformation.

The properties and equations used to calculate these properties (Refs 11 and 24) are discussed in the following sections.

Tensile Strength

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

The ultimate tensile strength was calculated using the following relationships 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} = the maximum load carried by the specimen, lb, and

t = the thickness 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

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

where

ν = static Poisson's ratio, and

DR = deformation ratio, 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 a sharp inflection point in the load-deformation curves, which 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 (Ref 2).

The equation used to calculate the static modulus was

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

where

E_s = static modulus of elasticity, psi, and

S_h = slope of the relationship between axial load and horizontal deformation, i.e., the ratio of axial load to horizontal deformation within the linear range, 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 4).

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 3) V_R and H_R for the loading cycle corresponding to $0.5\ N_f$. The equations are the same as those used for the static Poisson's ratio; however, since the relationships between load and deformation are essentially linear, the equations have 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 3.

The values of resilient Poisson's ratio were used to calculate resilient modulus of elasticity but were not analyzed. The values, however, are listed in Appendix D.

Resilient Modulus of Elasticity

The resilient modulus of elasticity was calculated using the resilient, or instantaneously recoverable, horizontal and vertical deformations which are more characteristic of the elastic deformations produced by repeated loads of short duration.

The equation used to calculate the resilient modulus was

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

where

E_R = resilient modulus of elasticity, psi, and

P_R = the applied repeated load, lb (Fig 3a).

Asphalt-Voids Ratio Density

The Asphalt-Voids Ratio, 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 345 kPa (500 psi). This is also referred to as the in-mold density and is used to calculate percent total air voids as defined by Test Method Tex-126-E. The specimen weight was determined after extrusion from the mold.

The AVR density, in pcf, was determined according to the following equation:

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

where

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

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

$$\text{Zero Air Voids Density (ZAVD)} = \frac{100\ \gamma_w}{\frac{P_s}{G_s} + \frac{P_a}{G_a}}$$

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 aggregate (obtained by performing Test Method Tex-109-E, Part IV, using the pressure pycnometer), and

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

The percent total air voids was determined from this relationship:

$$\text{Percent Total Air Voids} = 1 - \frac{\text{AVR density of specimen}}{\text{ZAVD}} \times 100 .$$

Permanent Deformation

The parameter selected as the basis for comparing the relative resistance to permanent deformation among the various specimens tested was permanent vertical deformation per cycle. This value was calculated as the slope of a straight line fitted by least squares regression to data points describing the relationship between permanent vertical deformation and number of load applications (Fig 4). Only the portion of the relationship between $0.10\ N_f$ and $0.70\ N_f$ was used. Several other parameters relating to permanent deformation characteristics were investigated and found to be of little value.

For the purpose of predicting permanent deformations in the field, permanent vertical 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.

TESTING PROGRAM

The variables included in this study were aggregate type, soil binder content, asphalt content, and moisture content. These variables were studied according to the testing program outlined in Figs 5 and 6. These tests were performed at room temperature, 24°C (75°F). For the repeated-load tests, two stress levels (Table 3) which would produce reasonable fatigue lives were selected. A limited number of mixtures were tested to evaluate the effects of moisture. These mixtures contained the optimum asphalt contents for maximum tensile strength.

Soil Binder Content, %	Type of Test	Stress Level	Eagle Lake Gravel - AC-20: 2.5	2.75	3.0	3.5	4.0	4.5	5.0	Lubbock Limestone - AC-10: 4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5
30	T_S	-	-	-	-	2	2	2	2										
	T_R	σ_L	-	-	-	3	2	2	1										
		σ_H	-	-	-	2	2	2	1										
25	T_S	-								-	-	-	-	2	4	3	4	2	2
	T_R	σ_L								-	-	-	-	-	1	3	3	2	-
		σ_H								-	-	-	-	-	1	3	3	2	-
20	T_S	-	-	-	2	4	2	2	-										
	T_R	σ_L	-	-	2	3	2	2	-										
		σ_H	-	-	2	3	2	3	-										
10	T_S	-	-	2	2	2	2	-	-	-	-	2	4	2	2	2	-	-	-
	T_R	σ_L	-	3	2	2	2	-	-	2	2	2	3	3	2	-	-	-	-
		σ_H	-	3	2	2	2	-	-	2	2	3	3	3	3	-	-	-	-
5	T_S	-	2	2	2	2	-	-	-	-	-	2	2	2	2	-	-	-	-
	T_R	σ_L	-	1	2	2	2	-	-	-	-	-	2	2	2	-	-	-	-
		σ_H	-	1	2	2	2	-	-	-	-	-	2	3	2	-	-	-	-
0	T_S	-	2	2	4	4	2	-	-	-	-	-	2	2	2	2	-	-	-
	T_R	σ_L	-	-	3	3	2	2	-	-	-	-	-	2	3	2	-	-	-
		σ_H	-	-	3	3	2	2	-	-	-	-	-	2	3	2	-	-	-

Fig 5. Summary of tests for Eagle Lake gravel and Lubbock limestone mixtures.

Soil Binder Content, %	Type of Test	Stress Level, kPa (psi)	Lubbock Limestone - AC-10: Asphalt Content, % 5.5	6.0	6.5	7.0
25	T_S	-	-	-	-	2
25	T_R	435 (63.1)	-	-	-	2
10	T_S	-	2	-	-	-
10	T_R	296 (42.9)	2	-	-	-
5	T_S	-	-	2	-	-
5	T_R	313 (54.1)	-	2	-	-
0	T_S	-	-	2	-	-
0	T_R	361 (52.3)	-	2	-	-

a.

T_S = static indirect tensile test

T_R = repeated-load indirect tensile test

Soil Binder Content, %	Type of Test	Stress Level, kPa (psi)	Eagle Lake Gravel AC-20: Asphalt Content, % 3.0	3.5	4.0
30	T_S	-	-	-	2
30	T_R	181 (26.2)	-	-	2
20	T_S	-	-	2	-
20	T_R	181 (26.2)	-	2	-
10	T_S	-	2	-	-
10	T_R	181 (26.2)	2	-	-
8	T_S	-	2	-	-
8	T_R	181 (26.2)	2	-	-
0	T_S	-	-	2	-
0	T_R	181 (26.2)	-	2	-

b.

Fig 6. Summary of tests for moisture damage of Eagle Lake gravel and Lubbock limestone mixtures.

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

Mixtures	Soil Binder Content, %	Stress Level, kPa (psi) Low σ_L	Stress Level, kPa (psi) High σ_H
Eagle Lake gravel	30	40 (5.8)	120 (17.4)
	20	40 (5.8)	120 (17.4)
	10	40 (5.8)	120 (17.4)
	5	80 (11.6)	120 (17.4)
	0	70 (10.2)	120 (17.4)
Lubbock limestone	25	150 (21.8)	250 (36.3)
	10	170 (24.7)	270 (39.2)
	5	200 (29.0)	300 (43.5)
	0	150 (21.8)	250 (36.3)

CHAPTER 3. ANALYSIS AND DISCUSSION OF TEST RESULTS

The purpose of this study was to investigate the effect of the amount of soil binder on the engineering properties of asphalt-treated materials. The following engineering properties were evaluated:

- General
  - Total air voids
  - Density
- Static Characteristics
  - Tensile strength
  - Static modulus of elasticity
- Repeated-Load Characteristics
  - Fatigue life
  - Resilient modulus of elasticity
  - Permanent deformation

The experimental approach was to determine the relationships between asphalt content and the above engineering properties and determine the optimum asphalt content for each property. These relationships and optimums were then evaluated with respect to soil binder content to determine whether properties could be improved by controlling the binder content. Finally, the effect of moisture on these relationships was evaluated.

AVR DESIGN OPTIMUM ASPHALT CONTENT

The total air voids were calculated using the in-mold AVR density and zero air void density as described in Chapter 2. The relationships between asphalt content and total air voids were determined for each aggregate gradation. From these relationships the laboratory AVR design optimum asphalt content for each aggregate gradation was determined according to Test Method Tex-126-E (Ref 17). The laboratory AVR design optimum asphalt contents are slightly greater than the asphalt contents corresponding to the inflection point on the straight line section of the AVR curves. The laboratory AVR design optimum asphalt contents, the corresponding total air voids, and the effect of the soil binder content are discussed in the following sections.

Eagle Lake Gravel

The relationships between asphalt content and total air voids are shown in Fig 7. These relationships indicate (1) that as the amount of soil binder decreased from 30 percent to 5 percent the total air voids decreased and (2) that the total air voids increased appreciably as the amount of soil binder decreased from 5 percent to 0 percent. It can be noted that the total air voids were approximately the same for binder contents of 5 and 10 percent. The relationships between soil binder content and total air voids at asphalt contents of 3.0, 3.5, and 4.0 percent are shown in Fig 8. The data for these curves were taken from Fig 7. These relationships indicate that the minimum total air voids occurred at a binder content of about 7 percent for mixtures containing 3.0 and 3.5 percent asphalt and at a binder content of about 10 percent for mixtures containing 4.0 percent asphalt.

An AVR design optimum asphalt content was determined for each binder content. It can be observed (Fig 9a) that the laboratory AVR design optimum asphalt content decreased from 4.5 percent for a 30 percent binder content to a minimum value of 3.5 percent for a 5 percent binder content and then increased slightly to 3.6 percent for zero percent binder content. The relationship in Fig 9b shows that the corresponding total air voids at laboratory AVR design optimum asphalt content remain constant at 1.6 percent for values of soil binder content ranging between 5 and 20 percent but increase appreciably for 0 percent and 30 percent binder contents.

Lubbock Limestone

The AVR curves for Lubbock limestone are shown in Fig 10, which suggests that, as the amount of soil binder decreased from 25 percent to 10 percent, the total air voids decreased to a minimum and then increased as the amount of soil binder decreased further from 10 to 0 percent. The relationships between soil binder content and total air voids at 5.5, 6.0, and 6.5 percent asphalt content are shown in Fig 11. The mixture containing 10 percent binder had the lowest total air voids regardless of the percentage of asphalt content.

The relationships between soil binder content and (a) laboratory AVR design optimum asphalt content and (b) total air voids are shown in Fig 12.

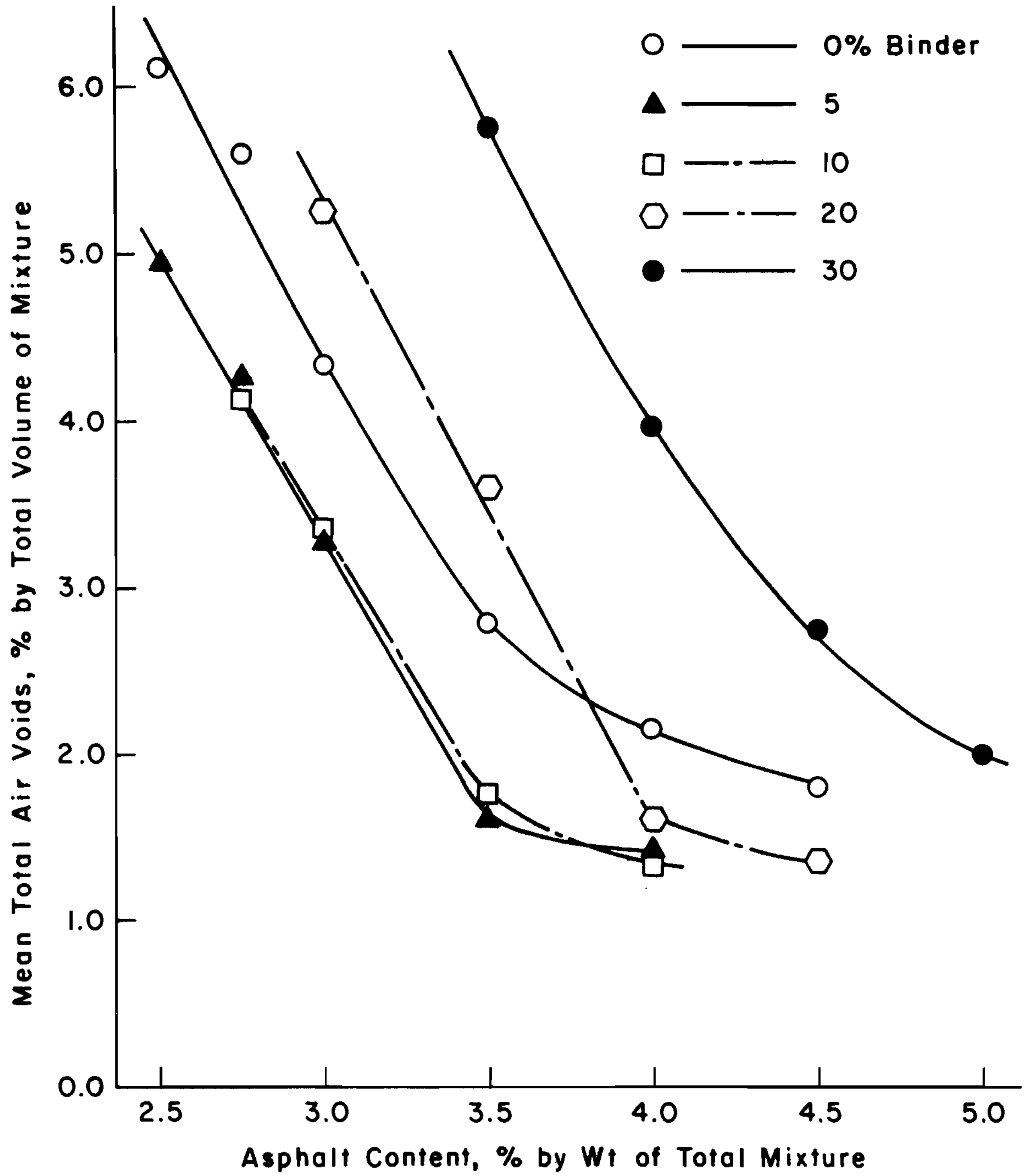


Fig 7. Relationships between asphalt content and total air voids for Eagle Lake gravel mixtures.

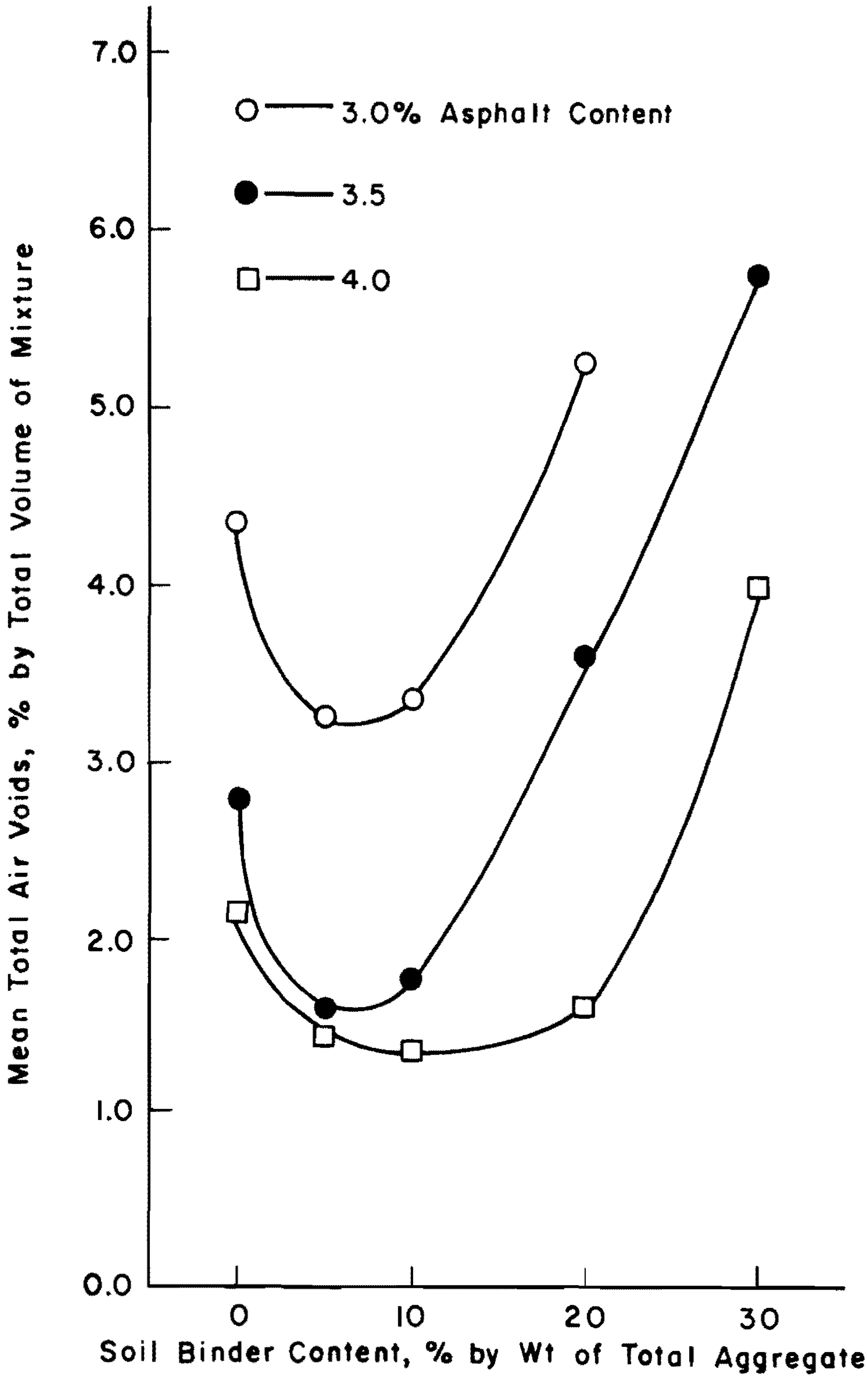


Fig 8. Relationships between soil binder content and total air voids for Eagle Lake gravel mixtures.

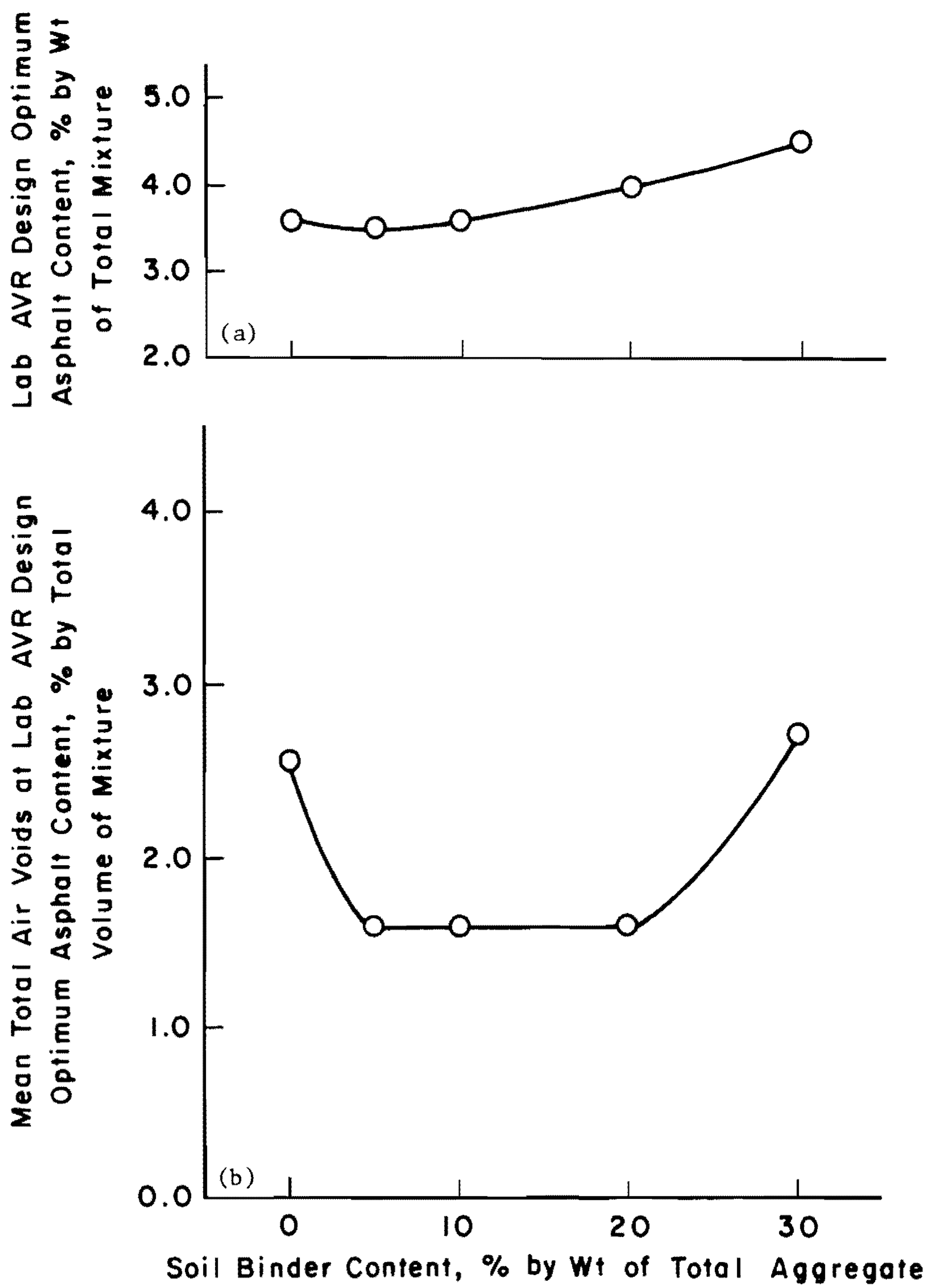


Fig 9. Relationships between soil binder content and (a) laboratory AVR design optimum asphalt content and (b) total air voids, for Eagle Lake gravel mixtures.

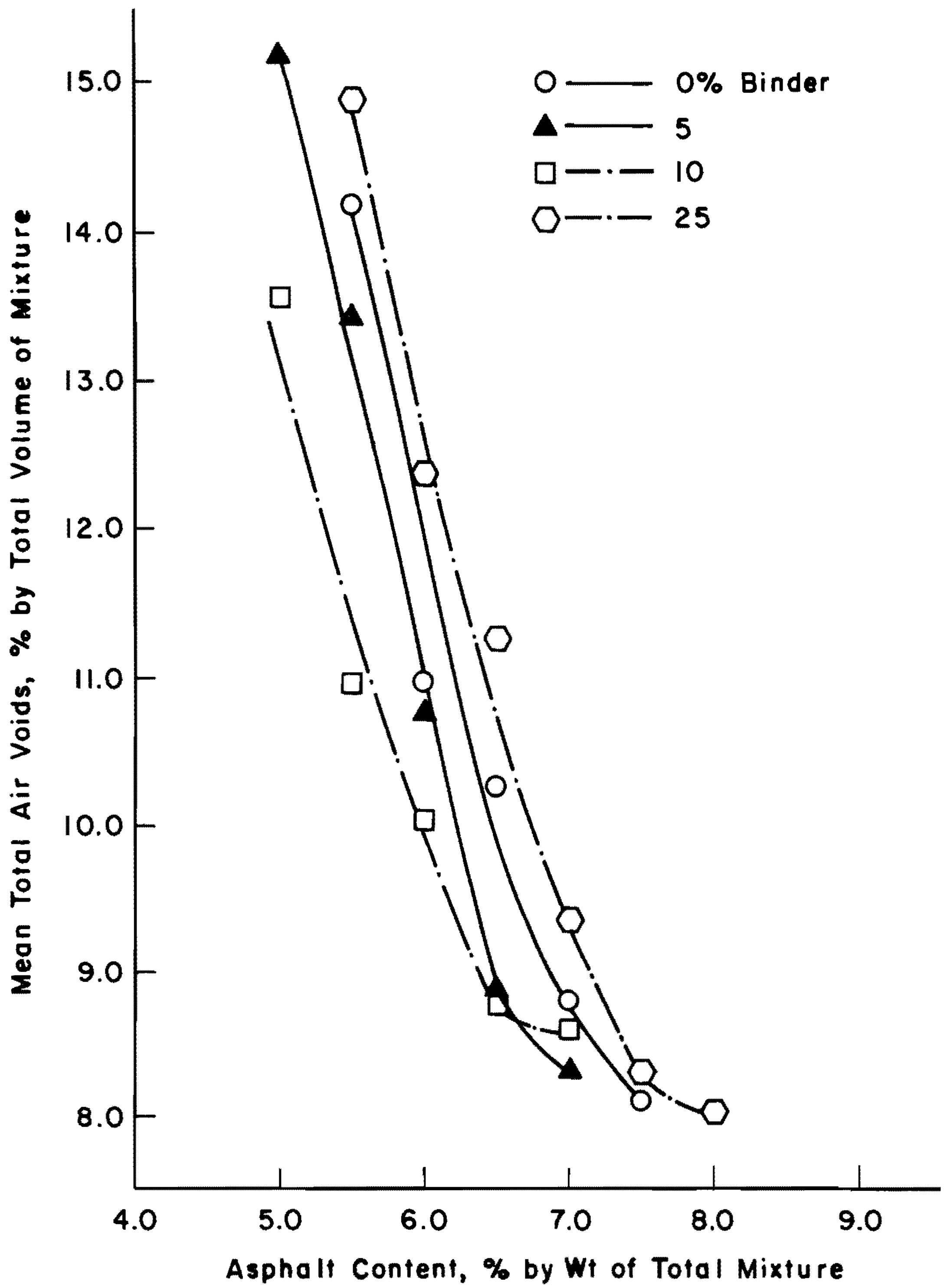


Fig 10. Relationships between asphalt content and total air voids for Lubbock limestone mixtures.

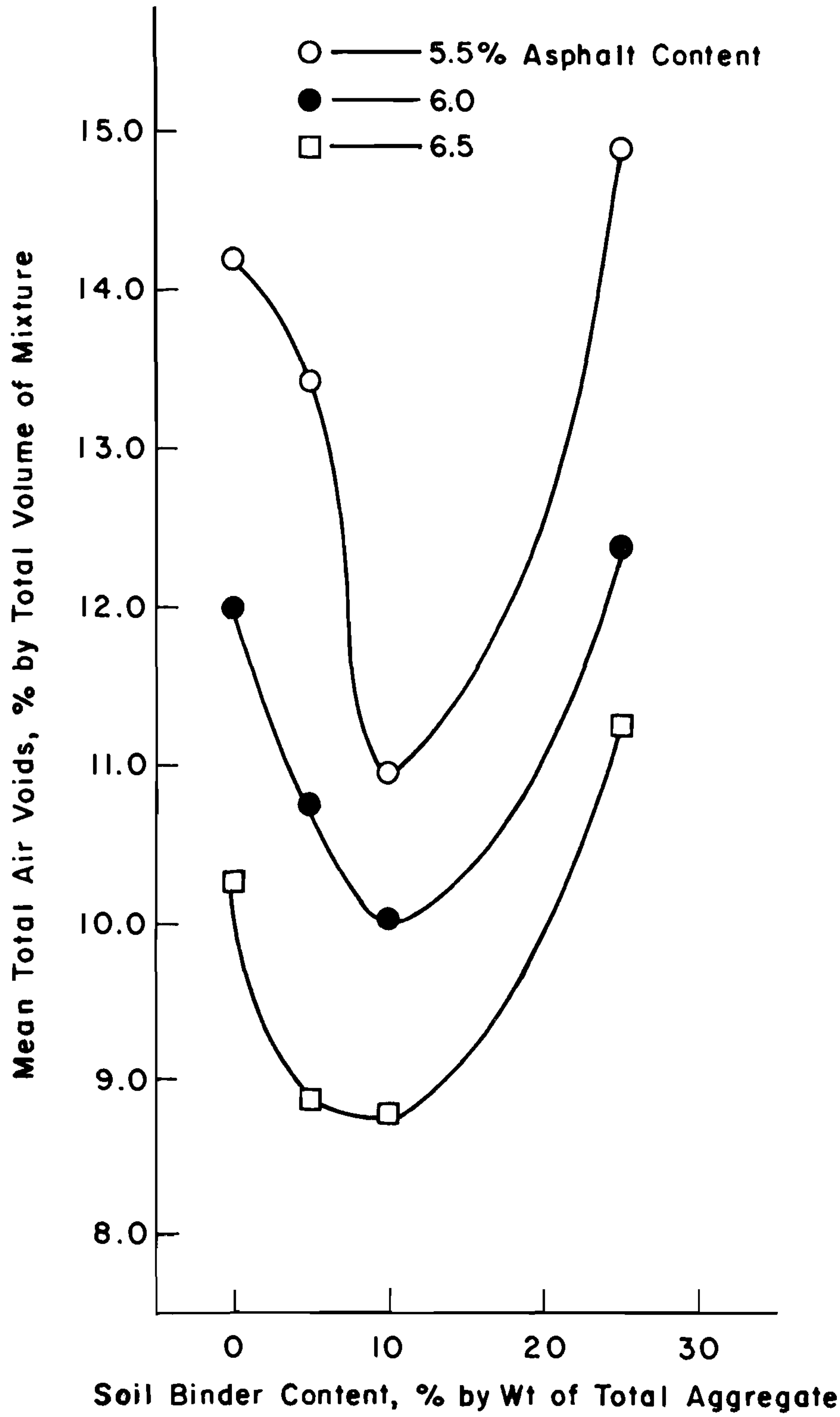


Fig 11. Relationships between soil binder content and total air voids for Lubbock limestone mixtures.

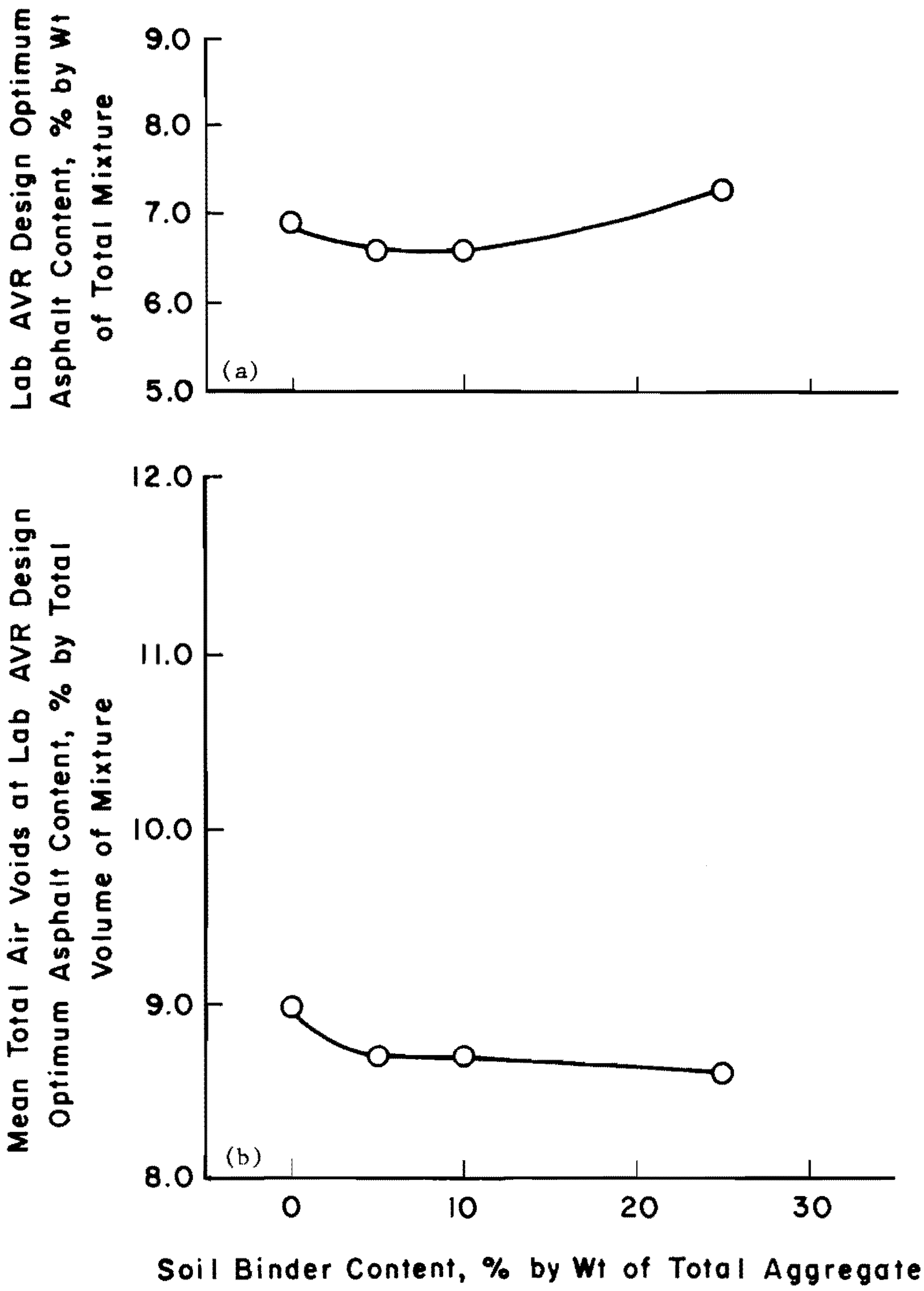


Fig 12. Relationships between soil binder content and (a) laboratory AVR design optimum asphalt content and (b) total air voids for Lubbock limestone mixtures.

Laboratory AVR design optimum asphalt contents were 6.6 percent for both 5 percent and 10 percent soil binder contents; however, the optimum asphalt contents are higher for 25 percent soil binder content (7.3 percent) and 0 percent soil binder content (6.9 percent). For each soil binder content, the total air voids at laboratory AVR design optimum asphalt content are very close, ranging from 9.0 percent for 0 percent soil binder content to 8.6 percent for 25 percent soil binder content (Fig 12b).

DENSITY

The relationships between asphalt content and in-mold AVR density for Eagle Lake gravel are shown in Fig 13. It is not shown, but the in-mold AVR densities were generally larger than the densities of the top and bottom sections of the specimens, an observation which had also been made in a previous study (Ref 24). From Fig 13 it can be seen that the mixture with 30 percent soil binder content had the lowest in-mold AVR density while the mixtures with 5 and 10 percent binder contents had the highest in-mold AVR densities.

The relationships between soil binder content and in-mold AVR density for mixtures with asphalt contents of 3.0, 3.5, and 4.0 percent and Eagle Lake gravel are illustrated in Fig 14. The optimum soil binder contents were about 7 percent for mixtures with 3.0 and 3.5 percent asphalt contents. The density curve for 4.0 percent asphalt content intersected the curve for 3.5 percent asphalt content at about 14 percent soil binder content. This indicates that the same density can probably be obtained by using either 3.5 or 4.0 percent asphalt content with 14 percent soil binder. At soil binder contents less that 14 percent, mixtures with 3.5 percent asphalt content had higher densities that those with 4.0 percent asphalt content; at soil binder contents higher than 14 percent, the reverse was true.

For the Eagle Lake gravel, the relationships between soil binder content and both the optimum asphalt content for in-mold AVR density and the maximum in-mold density are illustrated in Fig 15. It can be seen from these curves that the in-mold AVR density increased with decreased soil binder content, until it reached a maximum (2447 kg/m^3) at 5 percent soil binder content. Figure 15 also indicates that mixtures with 5 percent binder content had the lowest optimum asphalt content (3.5 percent) and the highest maximum density, 2447 kg/m^3 (153 pcf), and that the optimum asphalt content for in-mold AVR

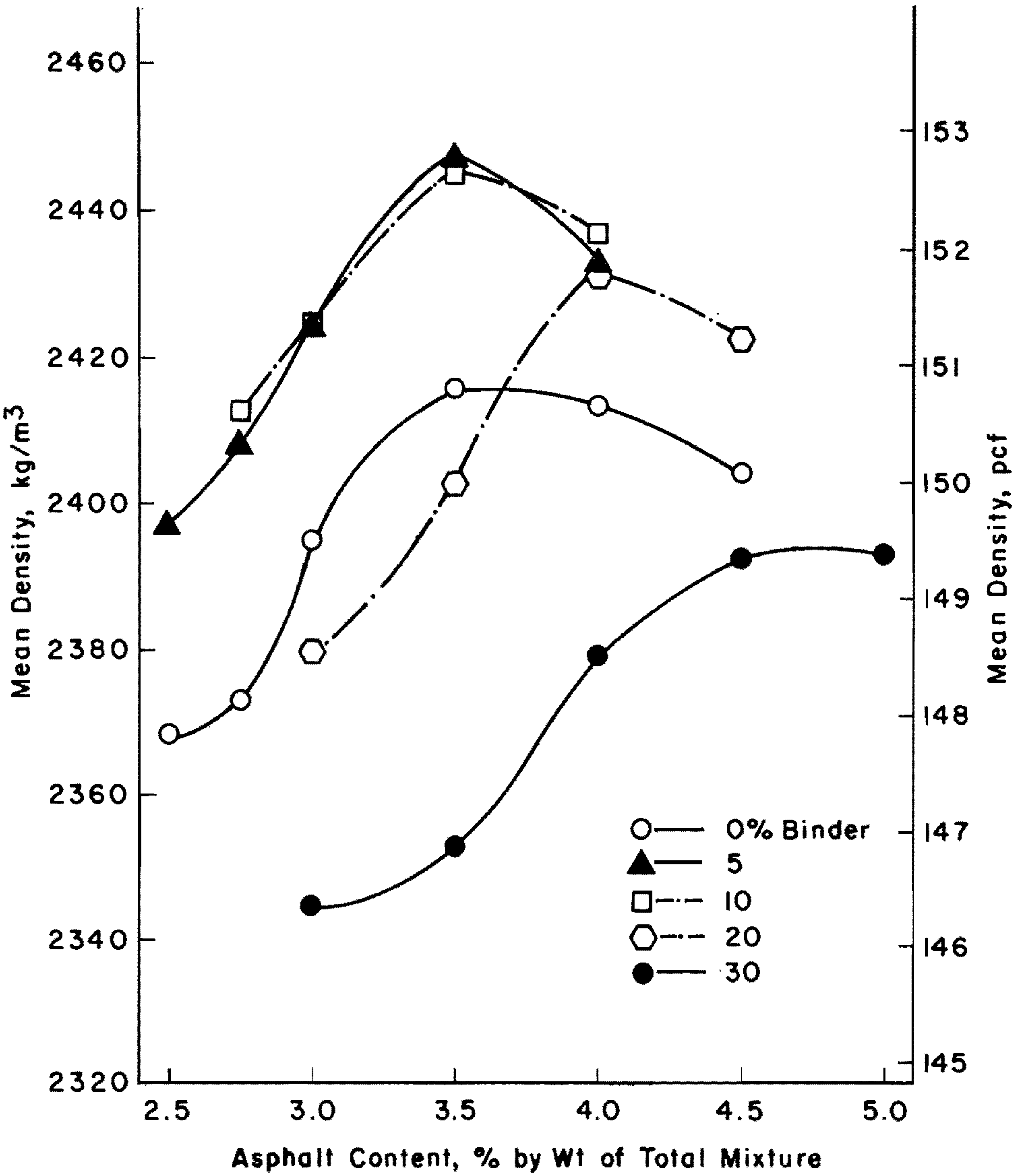


Fig 13. Relationships between asphalt content and in-mold AVR density for Eagle Lake gravel mixtures.

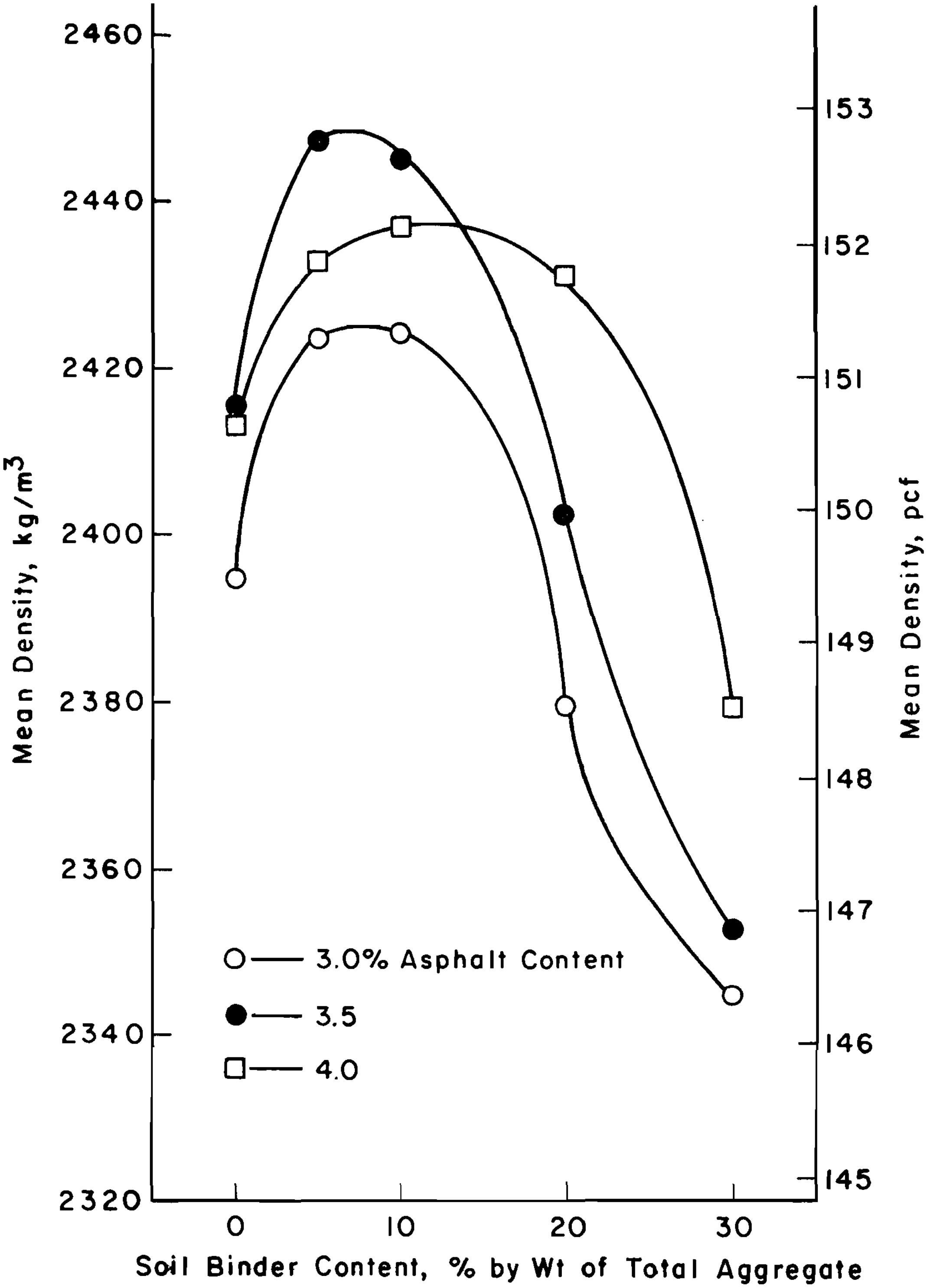


Fig 14. Relationships between soil binder content and both the optimum asphalt content and the corresponding in-mold AVR density for Eagle Lake gravel mixtures.

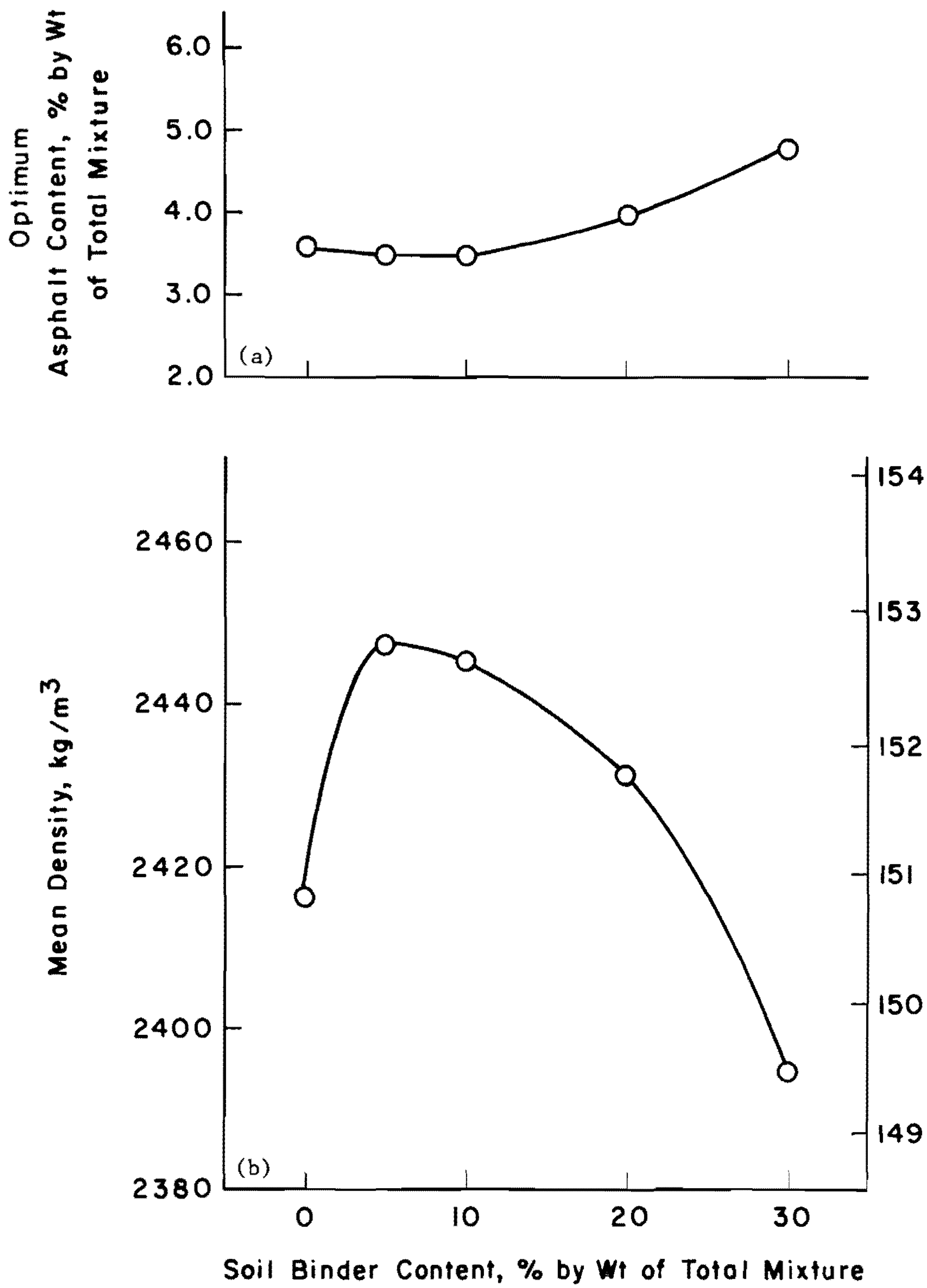


Fig 15. Relationships between soil binder content and (a) the optimum asphalt content for in-mold AVR density, and (b) the corresponding in-mold AVR maximum density for Eagle Lake gravel mixtures.

density (Fig 15a) was close to the laboratory AVR design optimum asphalt content (Fig 9a) for each binder content.

For Lubbock limestone, the relationship between asphalt content and in-mold AVR density is shown in Fig 16. As previously noted, the in-mold AVR densities were generally greater than the top and bottom densities and the density of the bottom specimens were generally greater than that of the top specimens. Figure 16 shows that the mixtures with 25 percent soil binder content had the lowest in-mold AVR density while the mixture with 10 percent soil binder had the highest in-mold AVR density. The optimum soil binder is about 10 percent for 5.5, 6.0, and 6.5 asphalt content and about 5 percent for 7.0 percent asphalt content (Fig 17).

The relationships between soil binder contents and both optimum asphalt content and the corresponding in-mold AVR maximum density for each soil binder are shown in Fig 18. The highest maximum density, 2220 kg/m^3 (139 pcf), occurred at about 8 percent soil binder and 6.5 percent optimum asphalt content. The optimum asphalt content for maximum in-mold AVR density decreased from 7.5 percent for 25 percent soil binder content to 6.5 percent for 10 and 5 percent soil binder contents and then increased to 7.3 percent for 0 percent soil binder content.

STATIC INDIRECT TENSILE TEST RESULTS

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

Tensile Strength

The effect of asphalt content on ultimate tensile strength was determined (Figs 19 and 20) and optimum asphalt contents were found for each soil binder content and each aggregate type. Values of optimum asphalt content for the Eagle Lake mixture ranged from 3.0 percent for soil binder contents of 5 and 10 percent to 4.0 percent for a binder content of 30 percent (Fig 21a), and from 5.5 percent for a binder content of 10 percent to 7.0 percent for a binder content of 25 percent for the Lubbock limestone mixtures (Fig 22a). The maximum tensile strength was 1,365 kPa (198 psi) (Fig 21b) for the

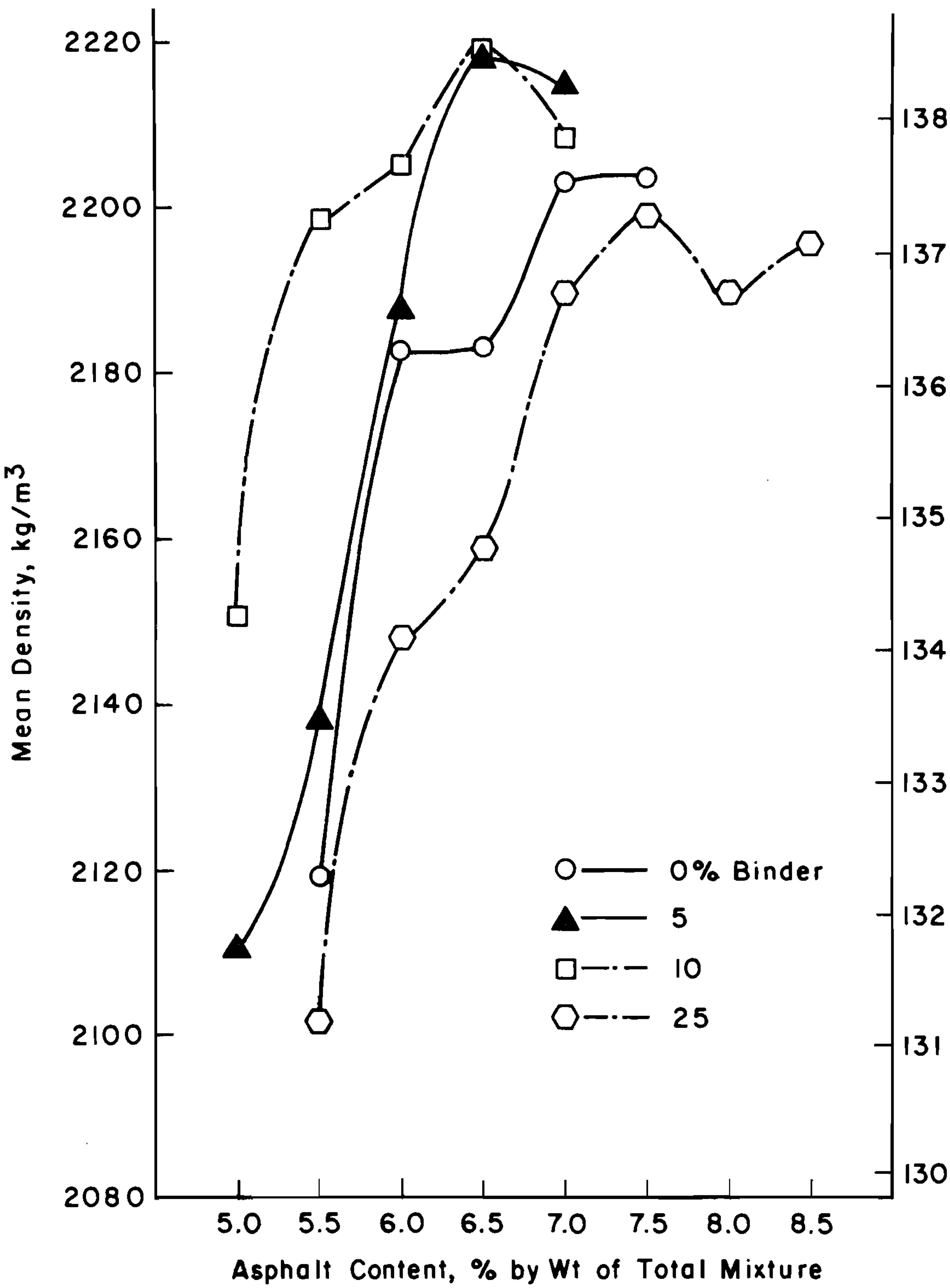


Fig 16. Relationships between asphalt content and in-mold AVR density for Lubbock limestone mixtures.

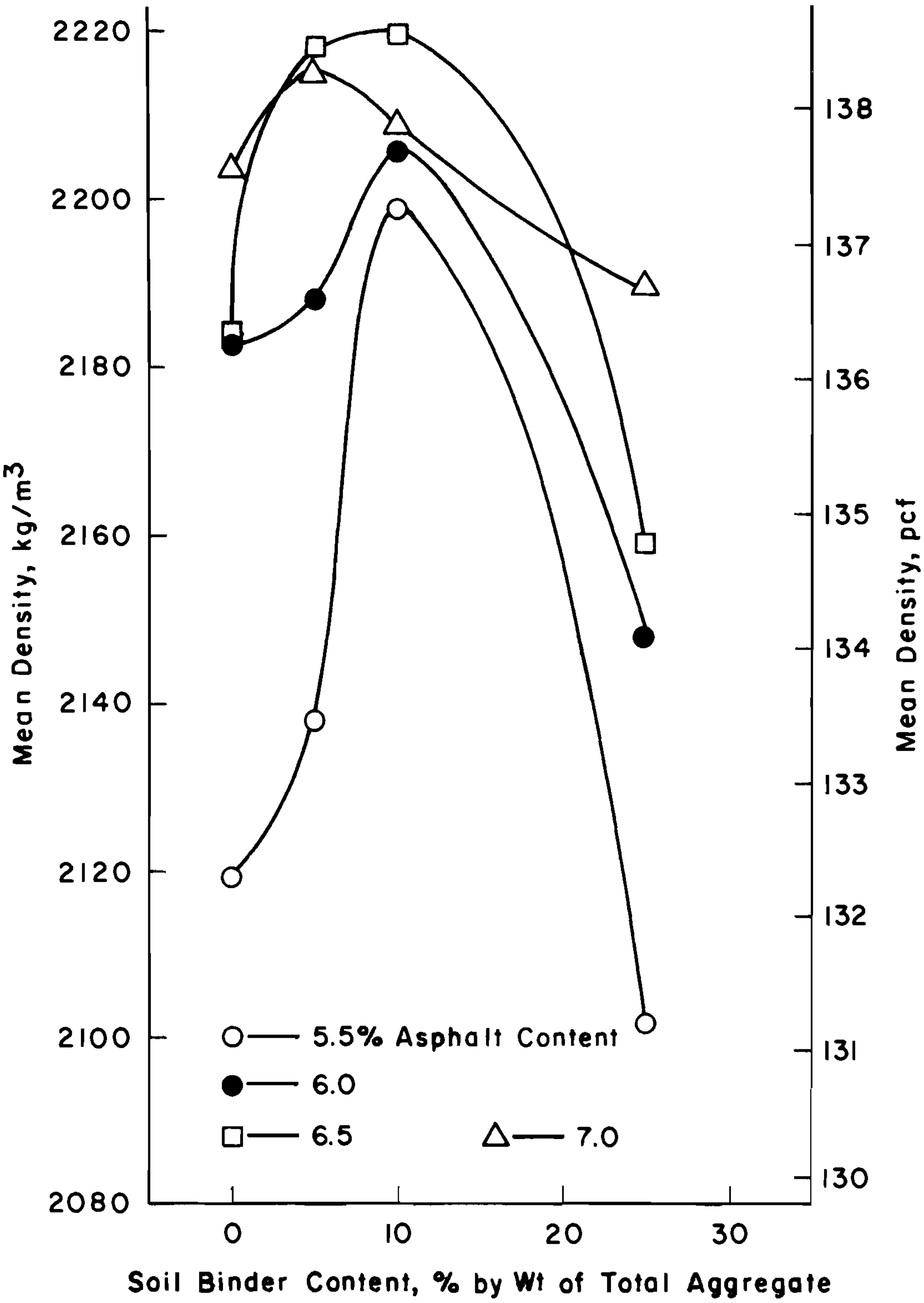


Fig 17. Relationships between binder content and in-mold AVR density for Lubbock limestone mixtures.

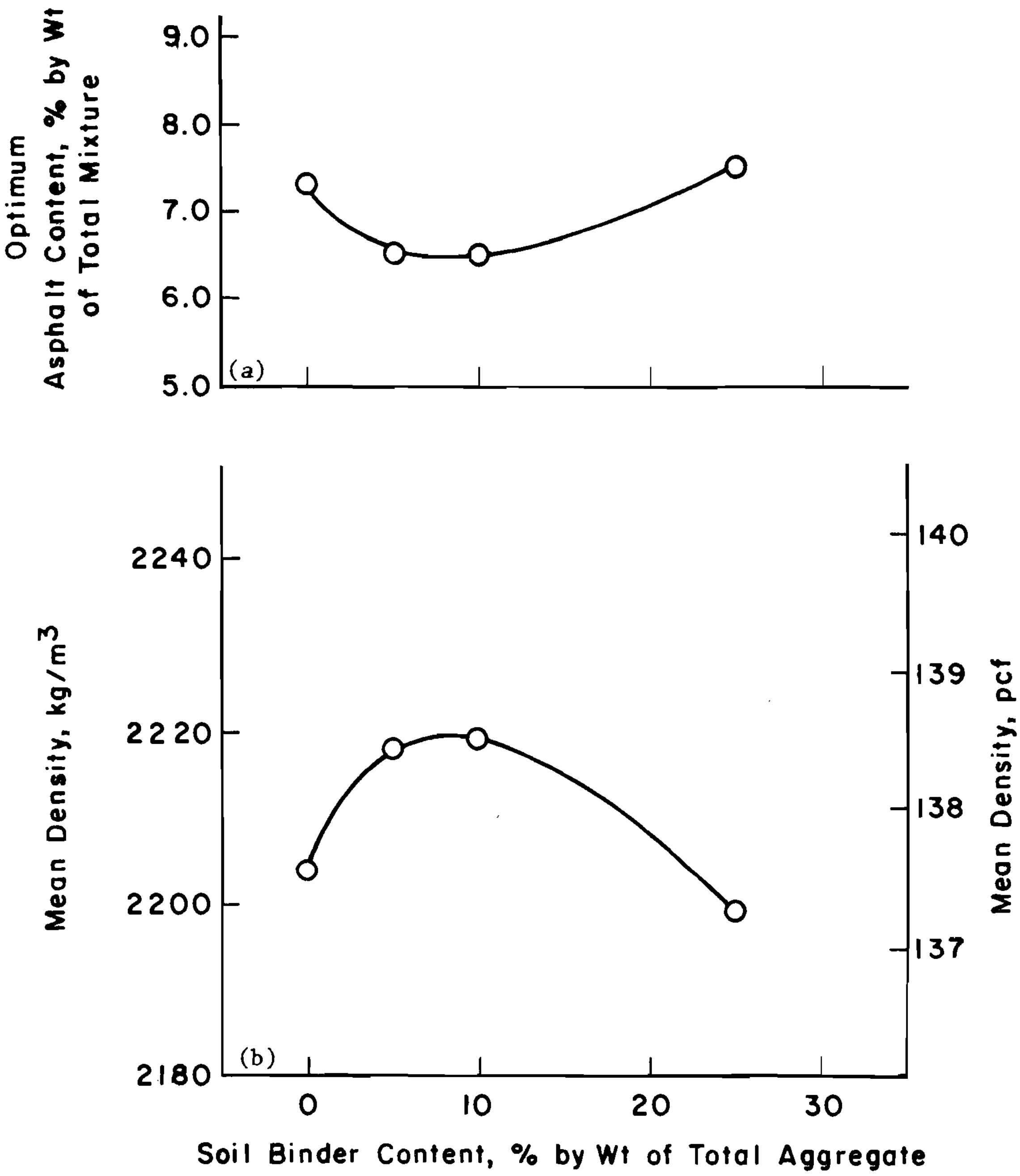


Fig 18. Relationship between soil binder content and (a) optimum asphalt content for in-mold AVR density, and (b) the corresponding in-mold AVR maximum density for Lubbock limestone mixtures.

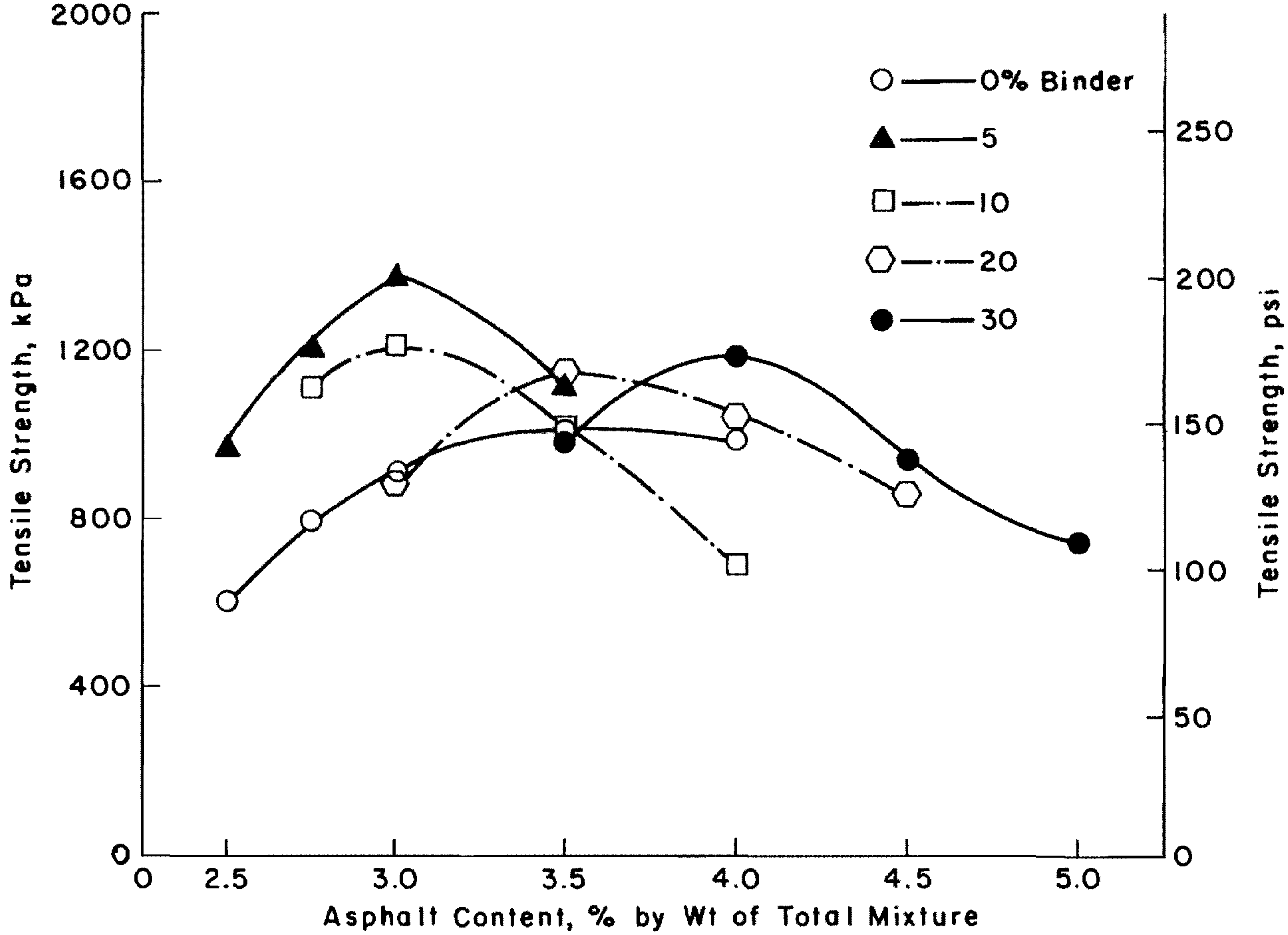


Fig 19. Relationships between asphalt content and tensile strength for Eagle Lake gravel mixtures.

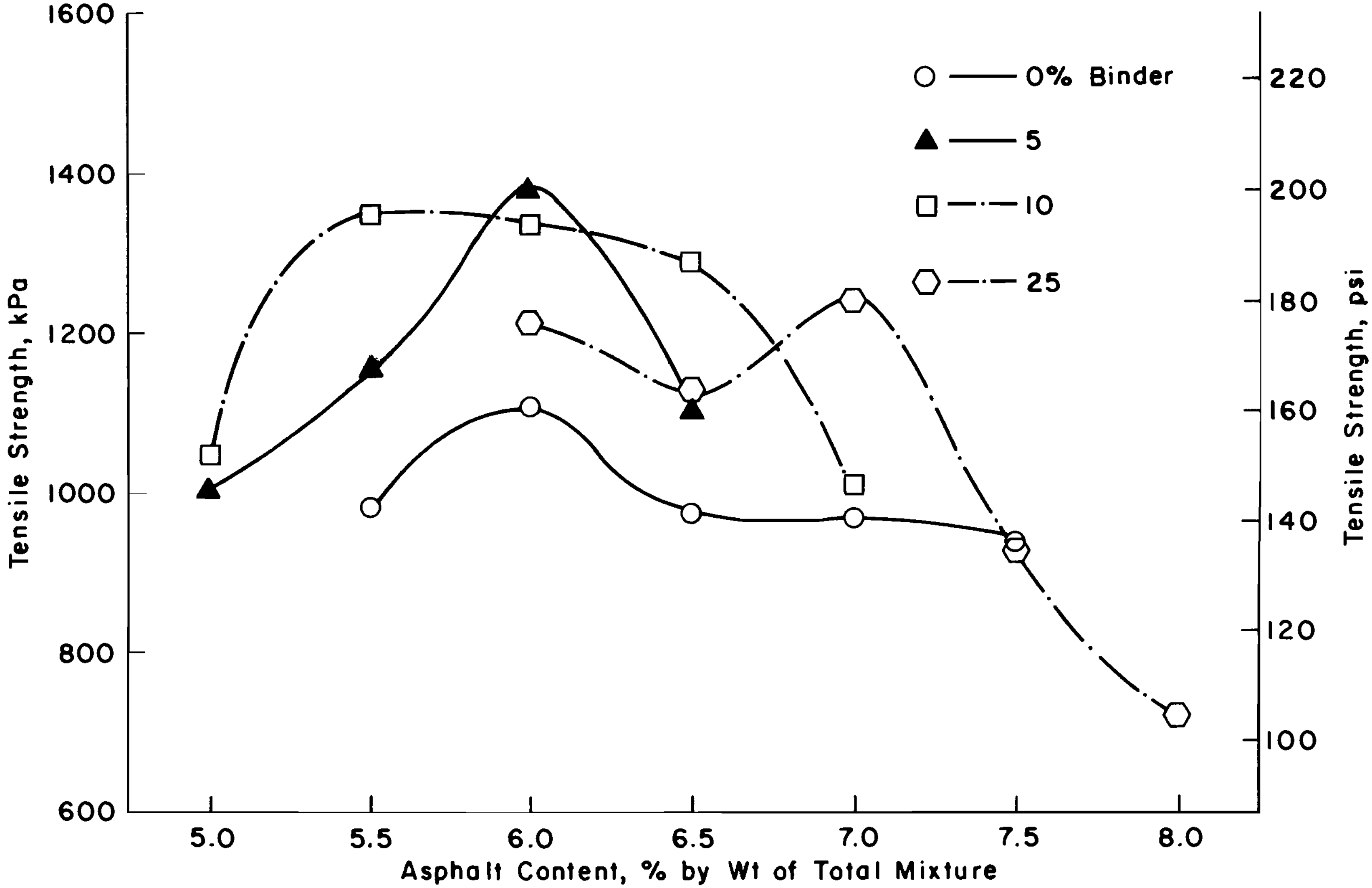


Fig 20. Relationships between asphalt content and tensile strength for Lubbock limestone mixtures.

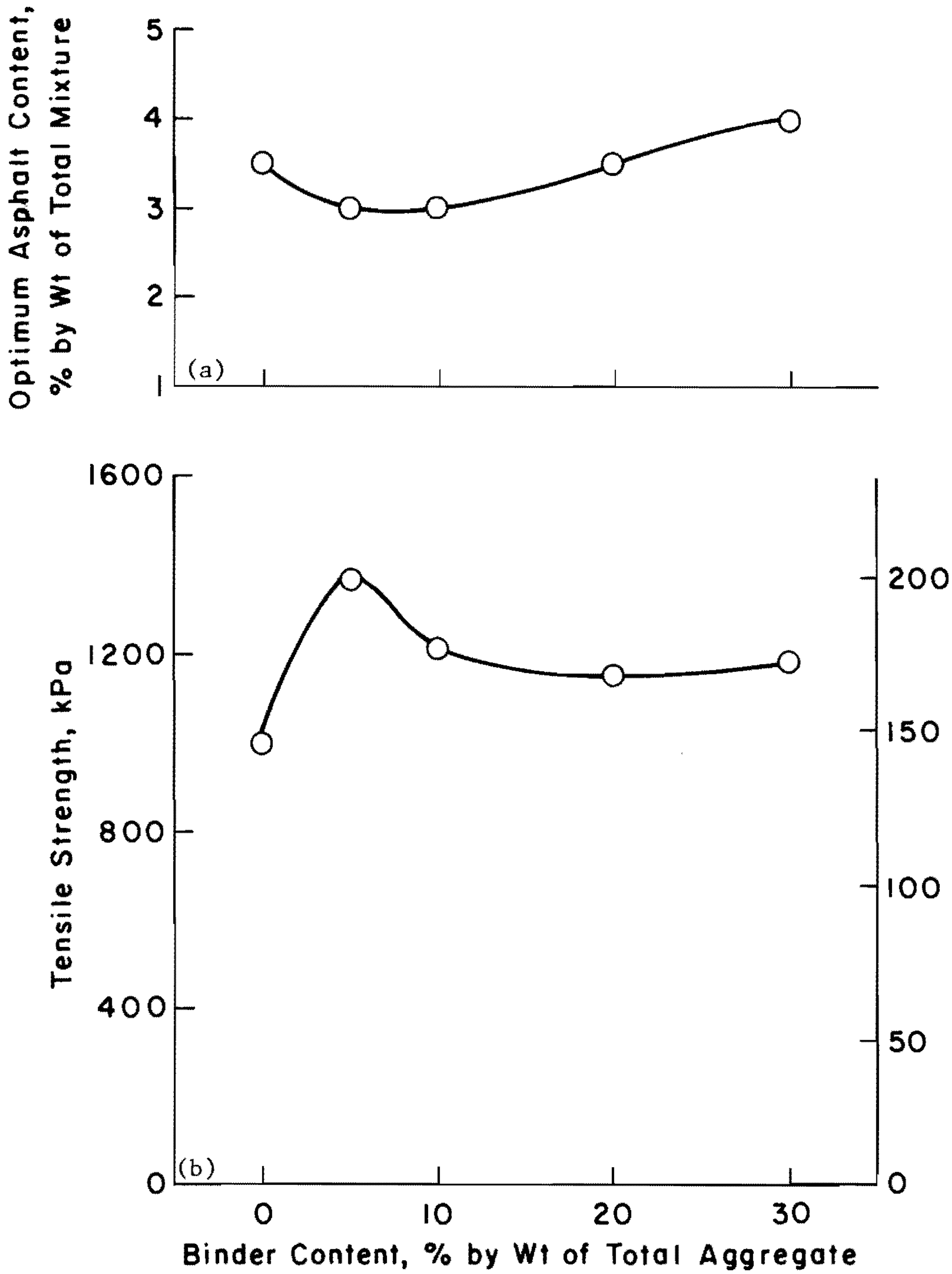


Fig 21. Relationships between binder content and both the optimum asphalt content and the corresponding maximum tensile strength for Eagle Lake gravel mixtures.

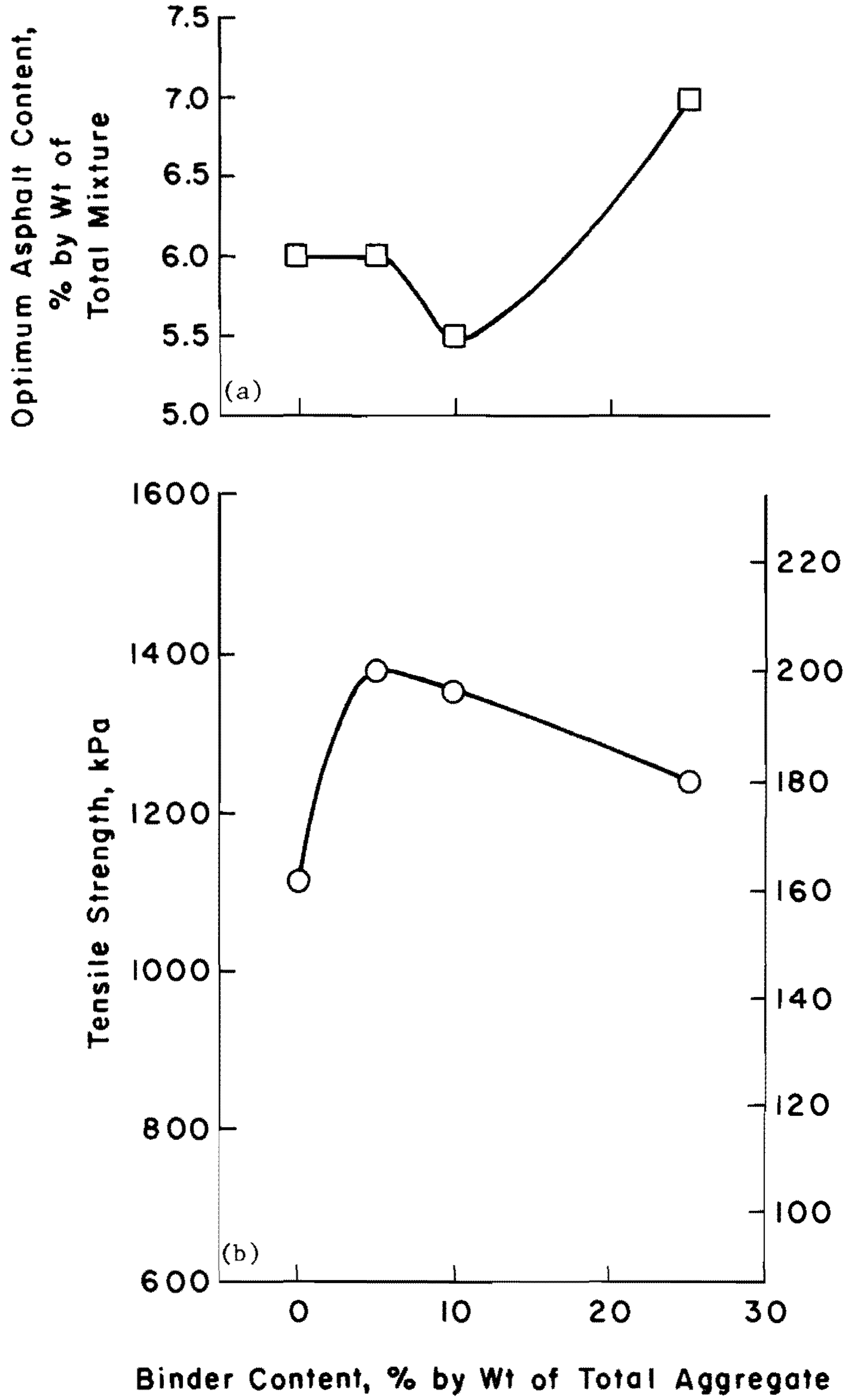


Fig 22. Relationships between binder content and both the optimum asphalt content and the corresponding maximum tensile strength for Lubbock limestone mixtures.

Eagle Lake gravel mixture and 1,367 kPa (200 psi) (Fig 22b) for the Lubbock limestone mixture, both of which occurred at a binder content of 5 percent.

The maximum tensile strengths of the Eagle Lake mixtures at binder contents of 30, 20, and 10 percent were essentially equal at about 1190 kPa (173 psi); however, the optimum asphalt contents were 4.0, 3.5, and 3.0 percent respectively (Fig 21). As the soil binder content decreased from 10 to 5 percent the strength increased by 180 kPa (26 psi) while the optimum asphalt content remained constant at 3.0 percent. Without any soil binder content, the ultimate tensile strength of the Eagle Lake gravel mixture decreased significantly, while the mixing optimum asphalt content increased from 3.0 to 3.5 percent.

Similar trends were found for the Lubbock limestone mixtures, except that the optimum asphalt contents for 5 and 0 percent binder contents (Fig 22) were the same (6.0 percent).

For the purpose of comparison, the relationships between binder content and tensile strength per 1 percent optimum asphalt content were evaluated (Fig 23). It can be seen that the Eagle Lake gravel mixture with 5 percent soil binder content produced the maximum ultimate tensile strength per unit percent of optimum asphalt content with a value of 456 kPa per one percent optimum asphalt content (66 psi per one percent optimum asphalt content) while the Lubbock limestone mixture with 10 percent binder content produced the maximum tensile strength per unit percent of optimum asphalt content with a value of 246 kPa per one percent optimum asphalt content (36 psi per one percent optimum asphalt content).

Static Modulus of Elasticity

The relationships between asphalt content and the static modulus of elasticity for Eagle Lake gravel and Lubbock limestone mixtures are shown in Figs 24 and 25 For all mixtures there were optimum asphalt contents for maximum static moduli of elasticity. These optimum asphalt contents for Eagle Lake gravel mixtures ranged from 3.0 percent for 0, 5, and 10 percent binder contents to 4.0 percent for 20 and 30 percent binder contents (Fig 26a). For Lubbock limestone mixtures the optimum ranged from 6.0 percent for 5 percent soil binder content to 7.0 percent for 25 percent soil binder content (Fig 27a).

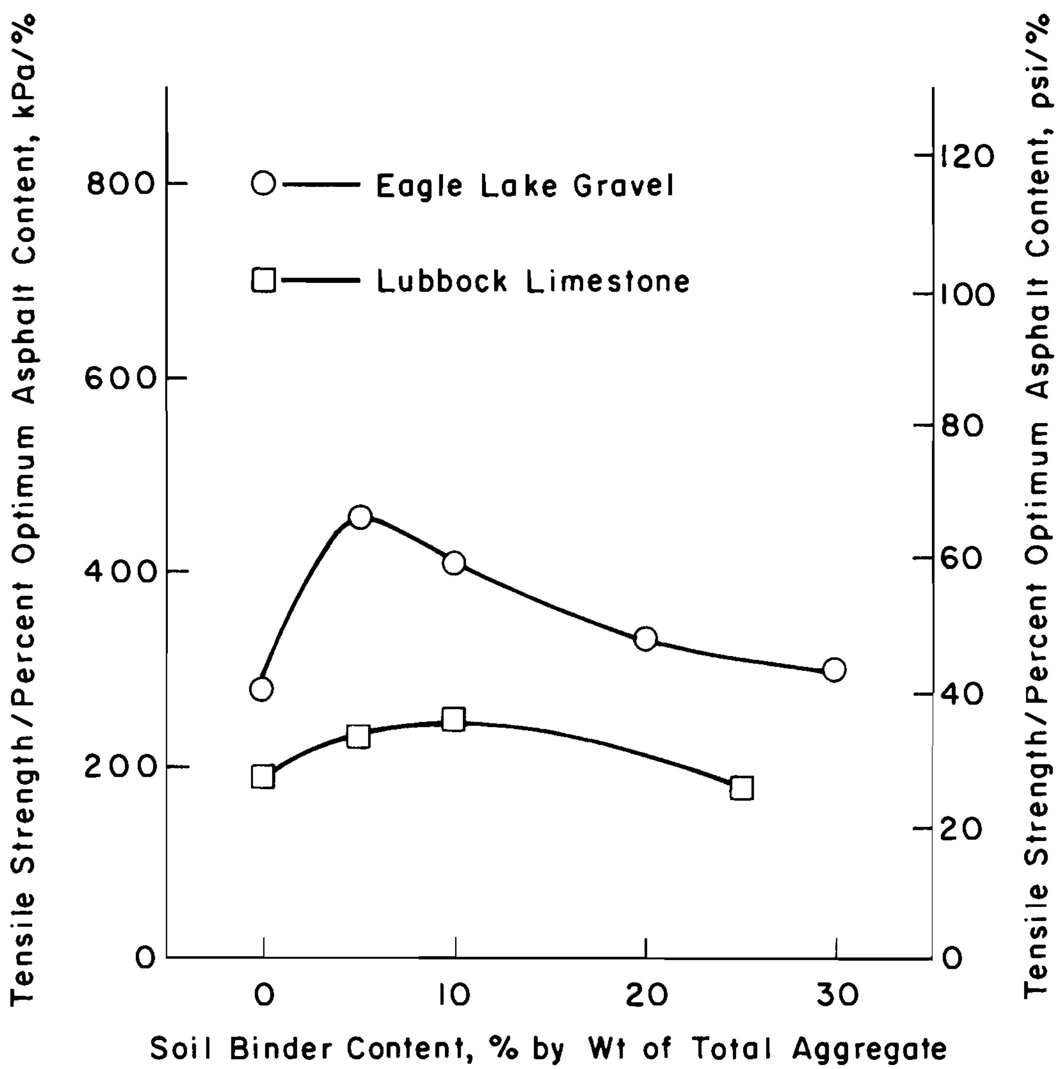


Fig 23. Relationship between binder content and the tensile strength per unit percent of optimum asphalt content for Eagle Lake gravel and Lubbock limestone mixtures.

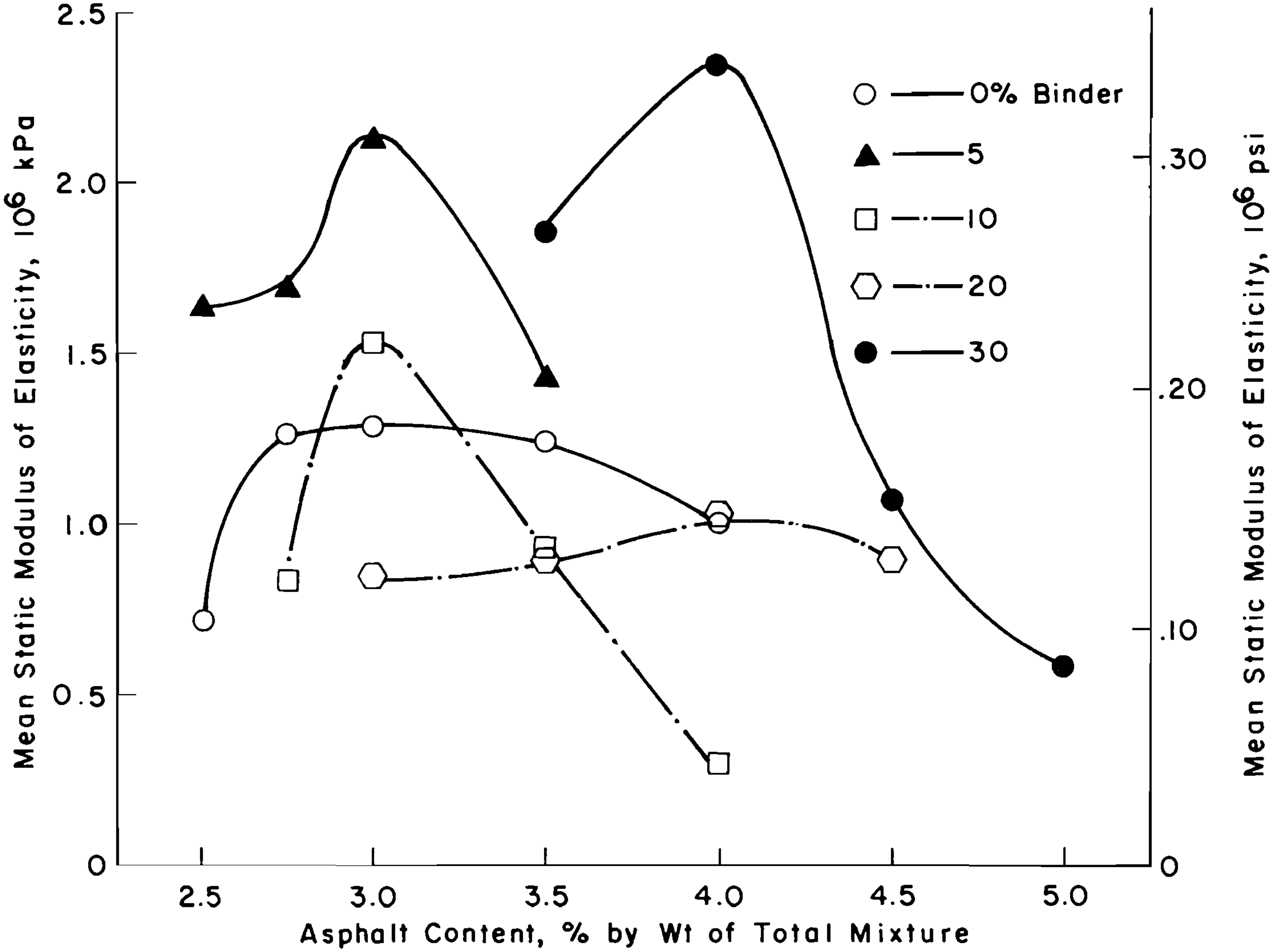


Fig 24. Relationships between asphalt content and static modulus of elasticity for Eagle Lake gravel mixtures.

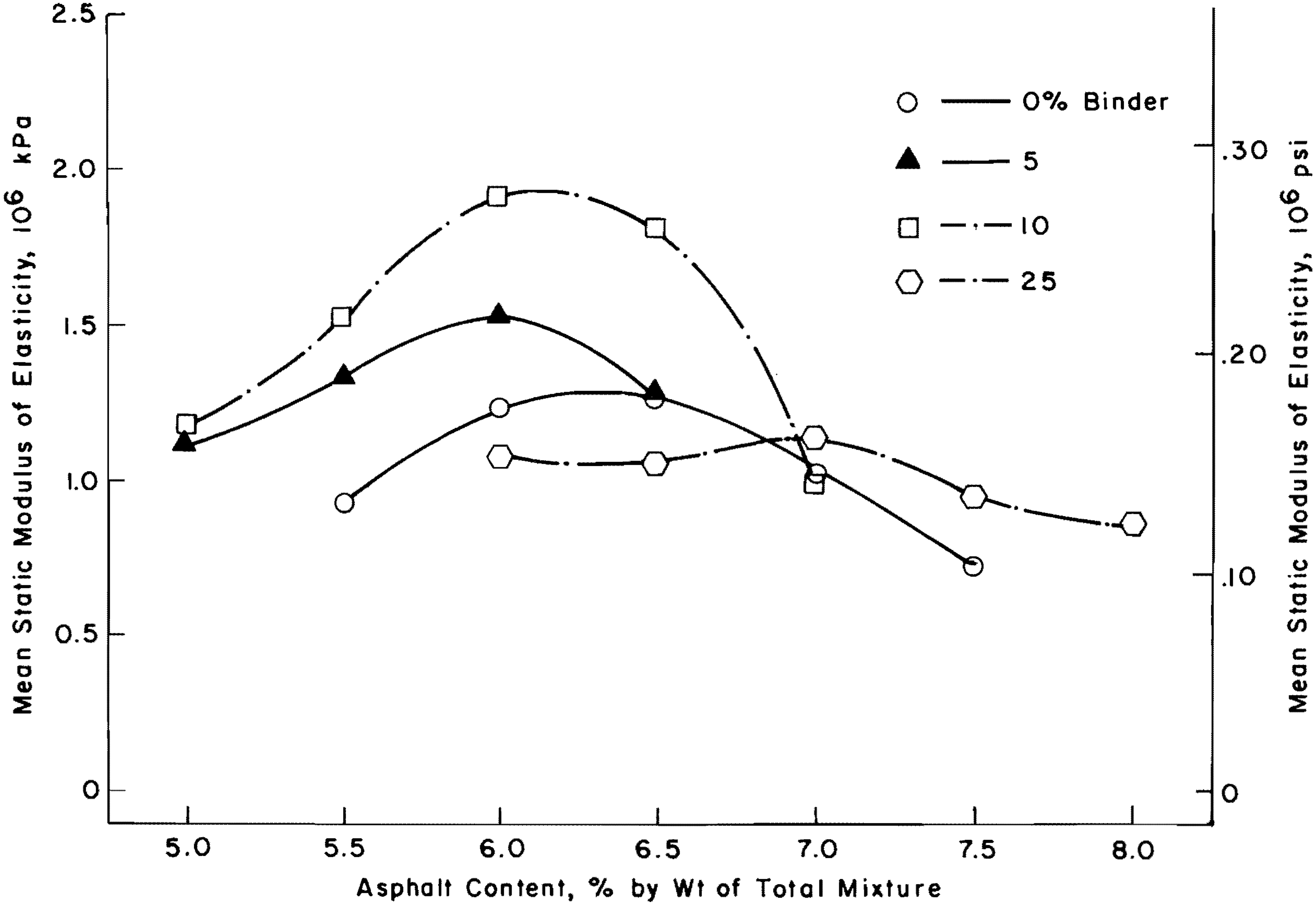


Fig 25. Relationships between asphalt content and static modulus of elasticity for Lubbock limestone mixtures.

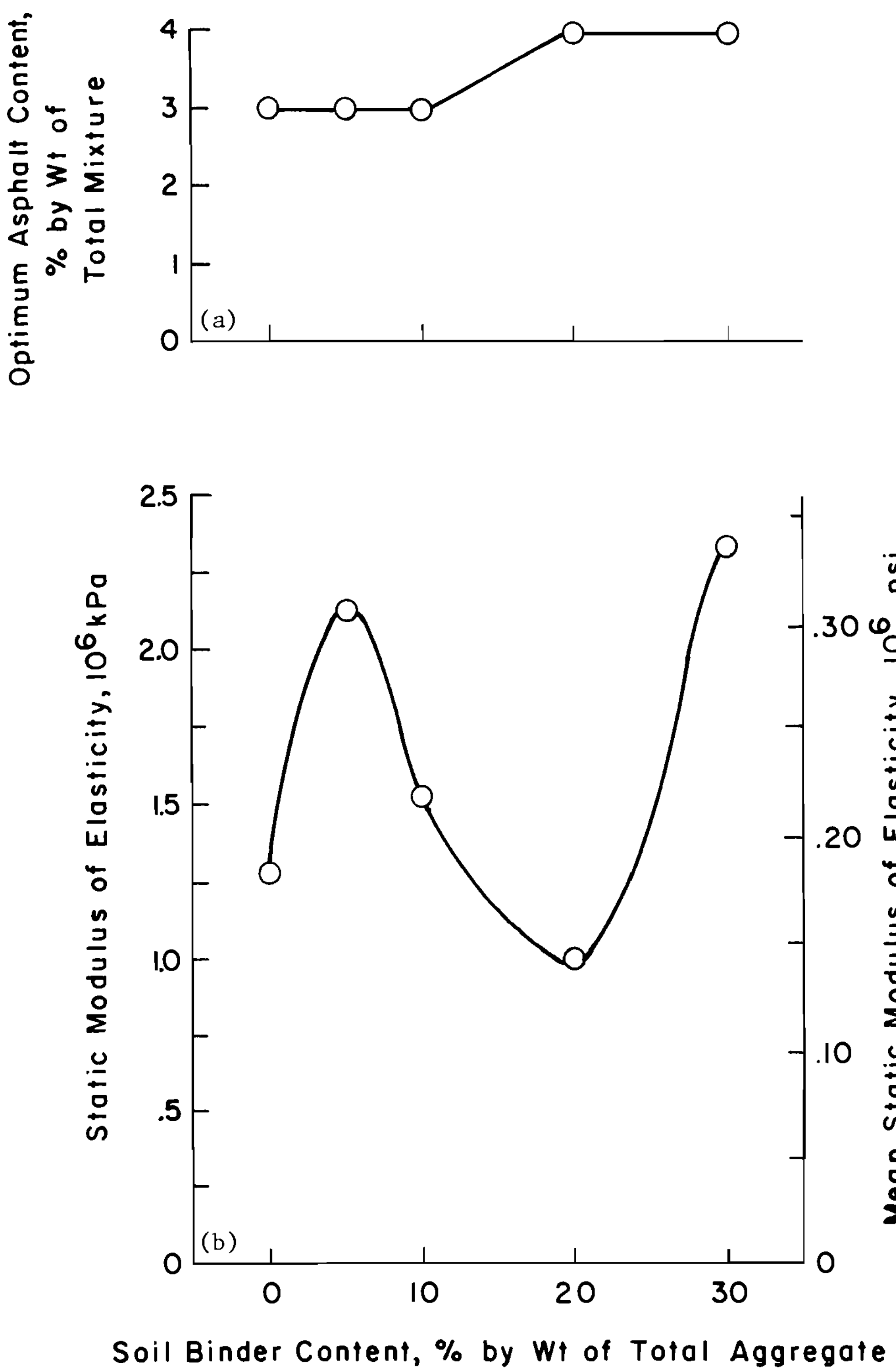


Fig 26. Relationship between soil binder content and both optimum asphalt content and the corresponding static modulus of elasticity for Eagle Lake gravel mixtures.

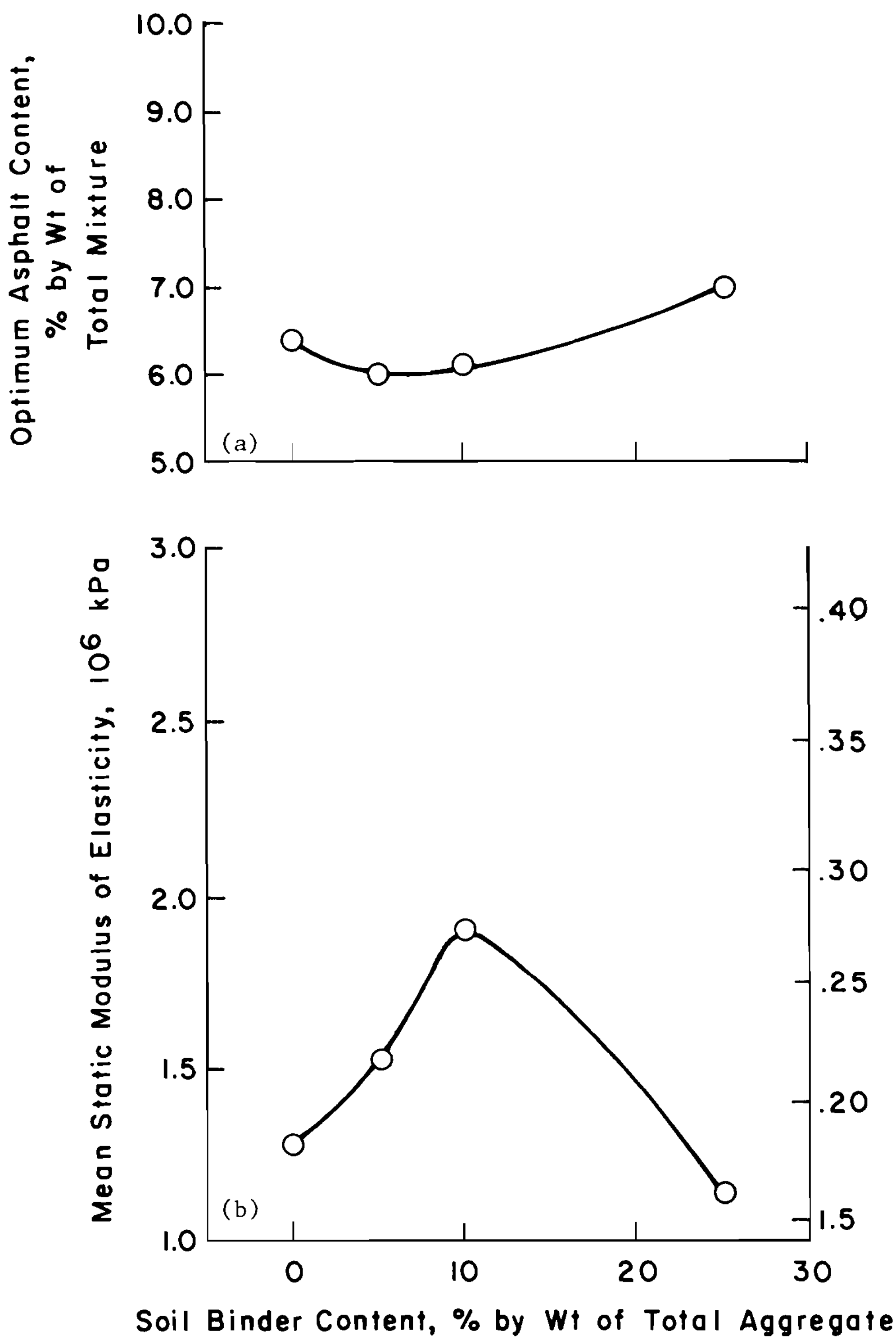


Fig 27. Relationships between soil binder content and both optimum asphalt content and the corresponding static modulus of elasticity for Lubbock limestone mixtures.

For the Eagle Lake gravel mixtures the maximum static modulus of elasticity occurred at 5 percent and 30 percent soil binder contents (Fig 26b). It is not clear, however, whether a true maximum occurred at 30 percent. The optimum binder content was found to be 10 percent for Lubbock limestone mixtures (Fig 27b).

Figure 28, which illustrates the relationships between soil binder content and modulus per one percent of optimum asphalt content, indicates a trend similar to that observed for tensile strength. The modulus per one percent optimum asphalt content was maximum at binder contents of 5 and 10 percent for Eagle Lake gravel and Lubbock limestone mixtures, respectively. Thus, in terms of economy of the mixture, the optimum binder contents would be 5 and 10 percent, which is the same as the optimum for maximum static modulus of elasticity.

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 for each of the mixtures of Eagle Lake gravel and Lubbock limestone. Results of repeated-load tests for individual specimens are presented in Appendix D.

Fatigue Life

For indirect tensile tests, stress difference was assumed equal to 4 times the tensile stress σ_T. In order to eliminate the effect of stress and to determine the effect of asphalt content and binder content, the fatigue life of the mixtures was evaluated for a tensile stress of 100 kPa (14.5 psi) for each binder content.

As shown in Figs 29 and 30, an optimum asphalt content for maximum fatigue life was found for each of the mixtures of Eagle Lake gravel and Lubbock limestone. It can be noted that these relationships were essentially symmetrical, i.e., the reduction in fatigue life wet of optimum was the same as that dry of optimum.

The relationships between soil binder content and the optimum asphalt contents are shown in Figs 31a and 32a. Optimum asphalt contents for the Eagle Lake mixtures ranged from 2.9 percent for 5 percent binder content to 4.6 percent for 30 percent binder content (Fig 31a) and from 4.5 percent

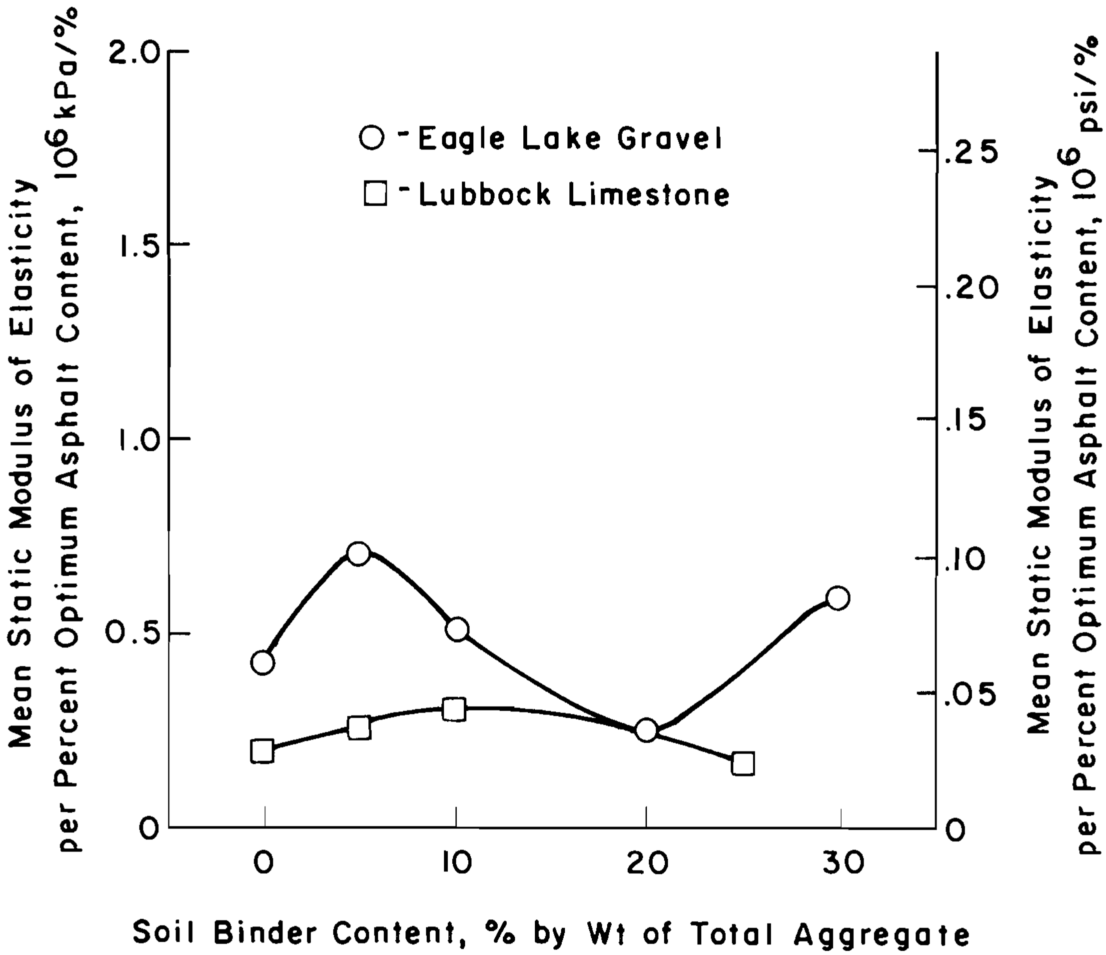


Fig 28. Relationship between binder content and the static modulus of elasticity per unit percent of optimum asphalt content for Eagle Lake gravel and Lubbock limestone mixtures.

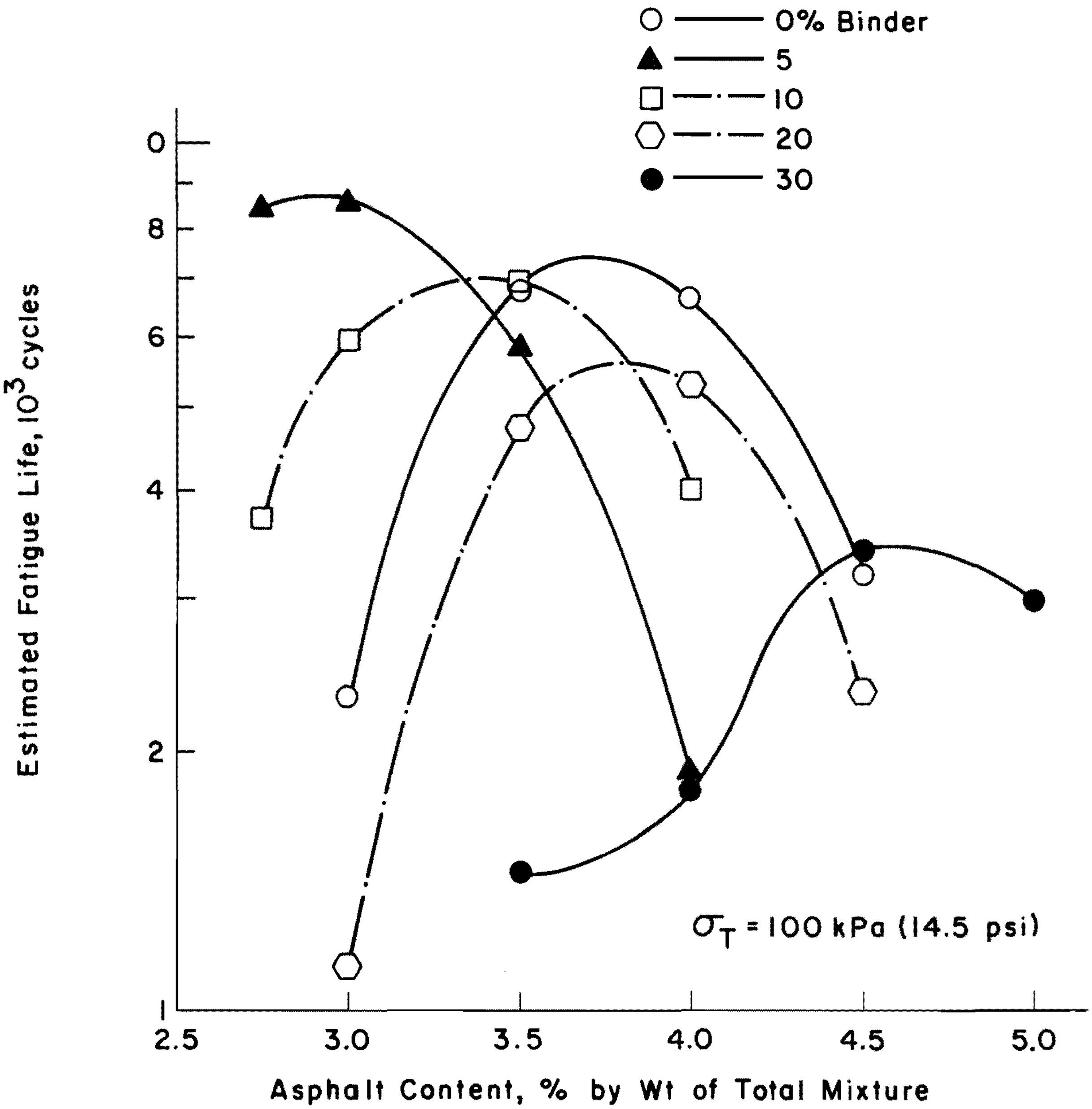


Fig 29. Relationships between asphalt content and fatigue life for Eagle Lake gravel mixtures.

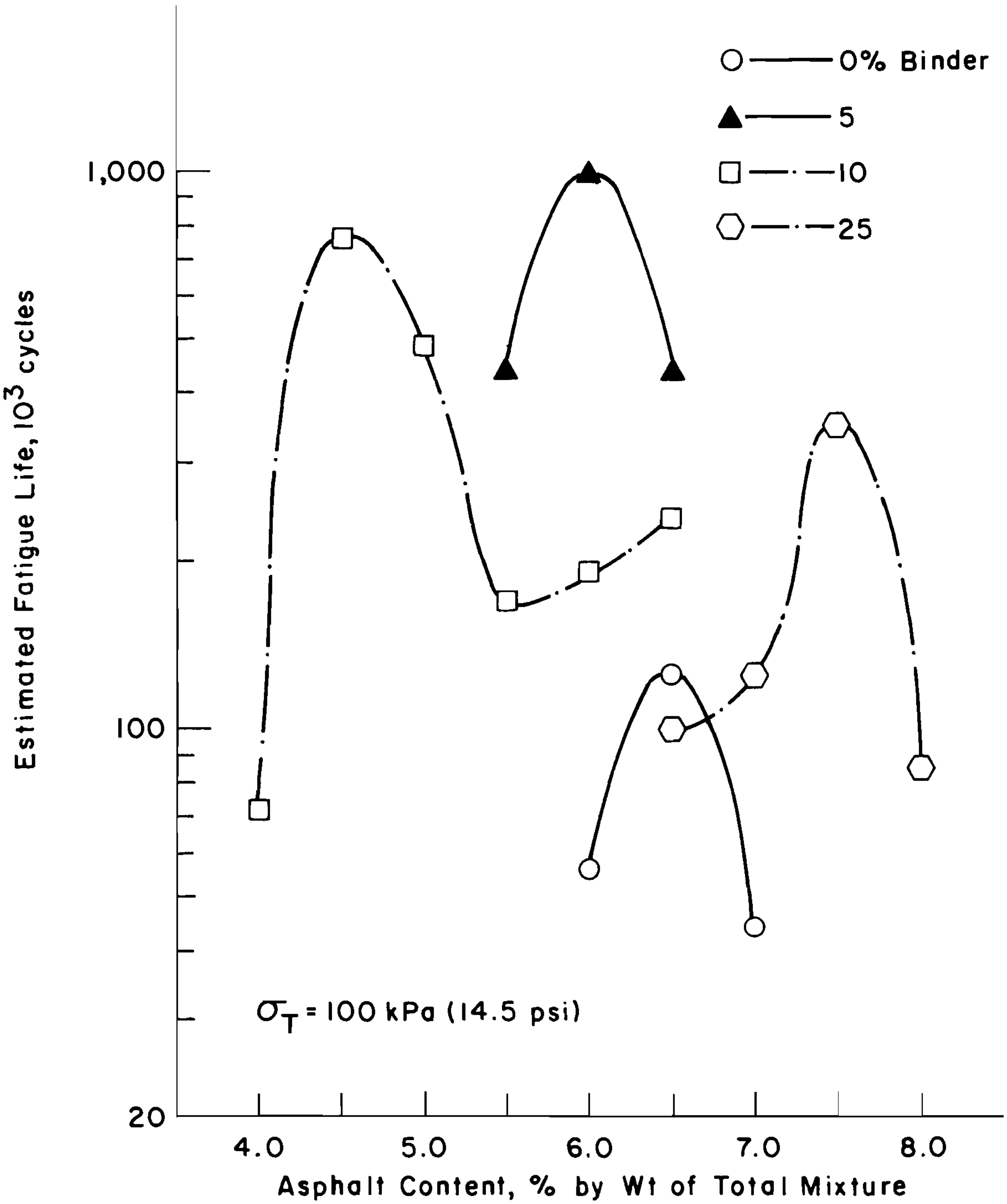


Fig 30. Relationships between asphalt content and fatigue life for Lubbock limestone mixtures.

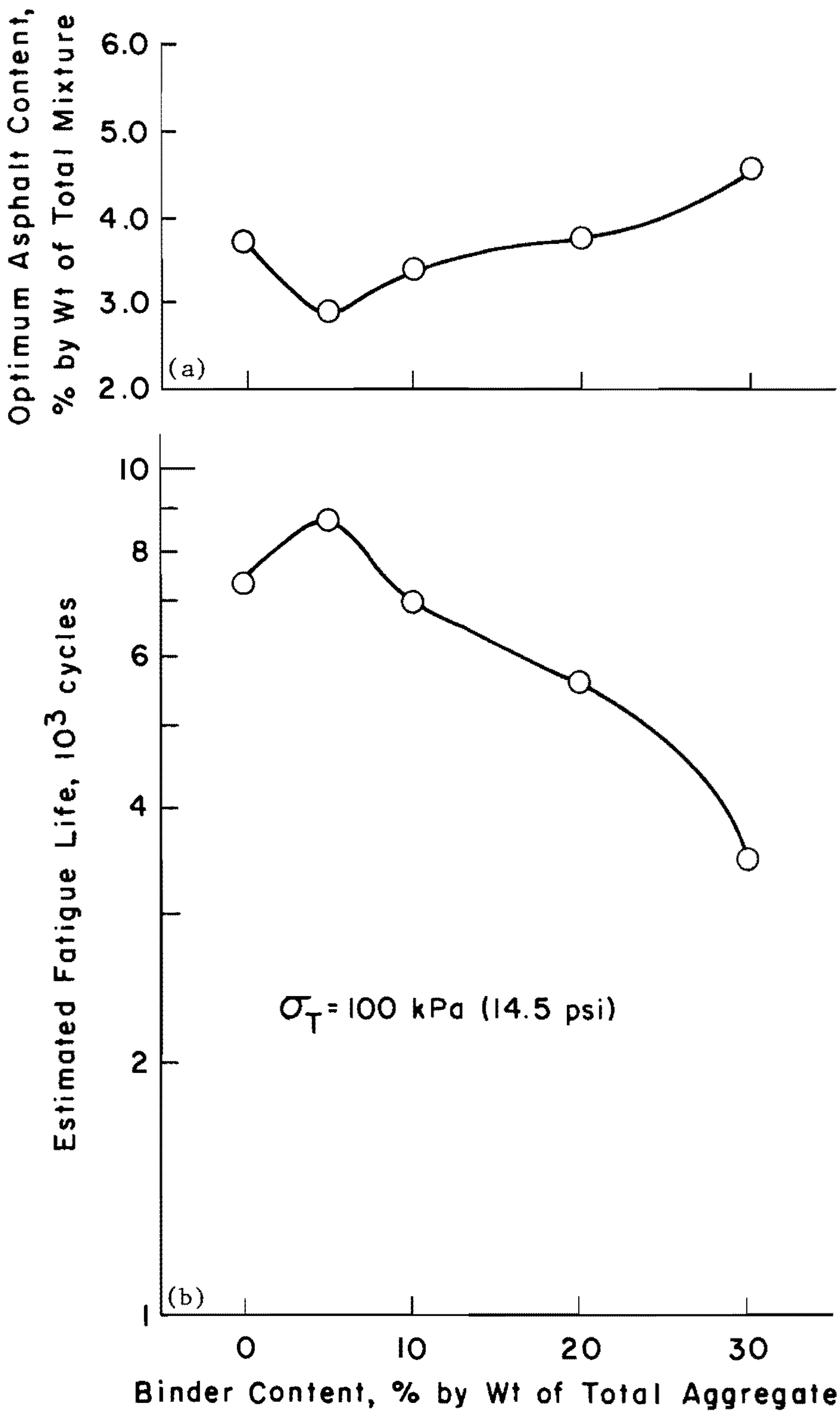


Fig 31. Relationships between binder content and both optimum asphalt content and the corresponding fatigue life for Eagle Lake mixtures.

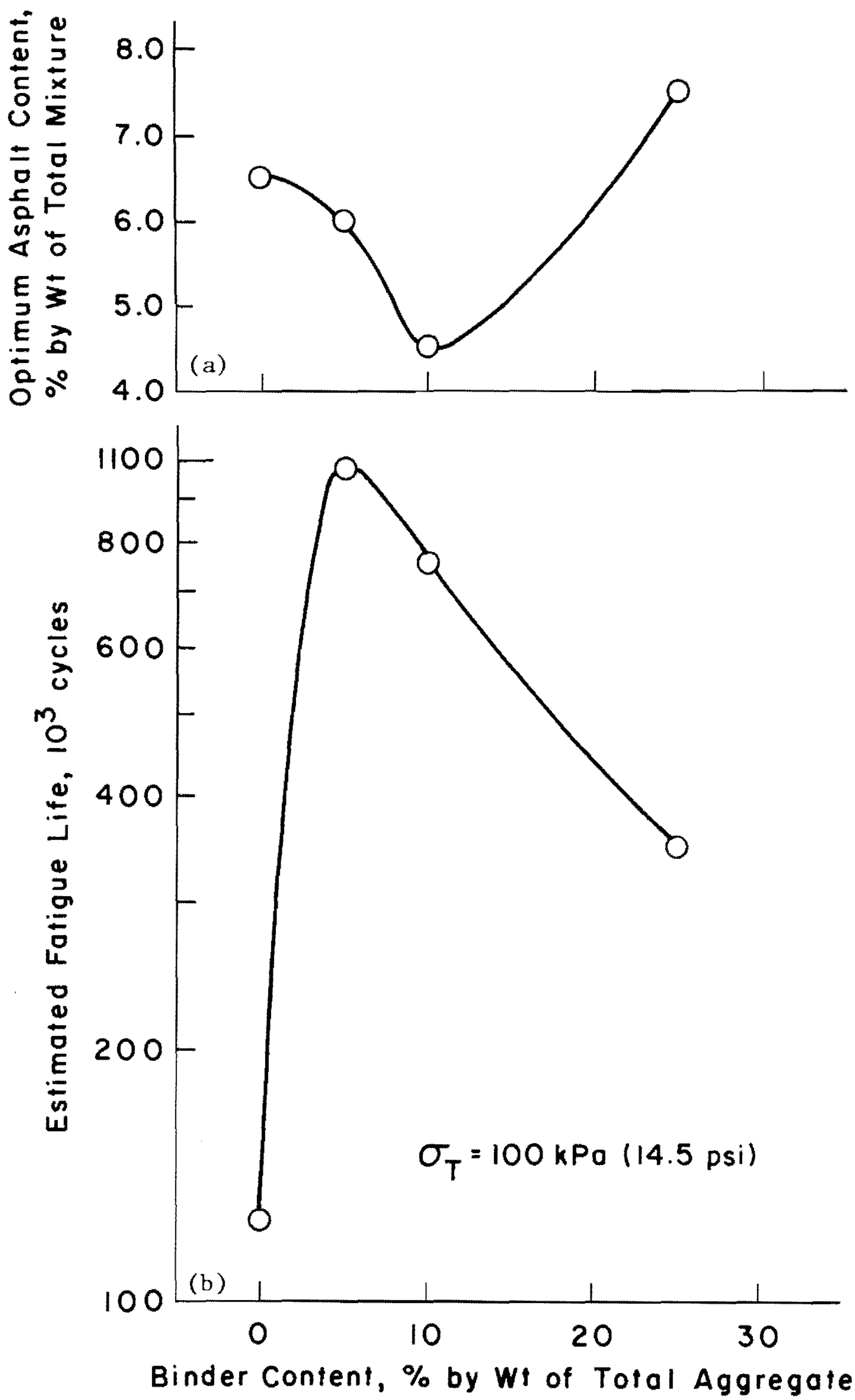


Fig 32. Relationships between soil binder content and both optimum asphalt content and the corresponding fatigue life for Lubbock limestone mixtures.

for 10 percent binder to 7.5 percent for 25 percent binder for the Lubbock limestone mixtures (Fig 32a). For both mixtures the optimum asphalt content for 0 percent soil binder content was higher than the optima for mixtures with 5 and 10 percent soil binder contents (Figs 31a and 32a).

The optimum soil binder content for maximum estimated fatigue life was 5 percent for both types of aggregate (Figs 31b and 32b). The maximum estimated fatigue life was about 8,700 cycles for the Eagle Lake gravel at 2.9 percent asphalt content and was about 980,000 cycles for the Lubbock limestone at 6.0 percent asphalt content.

The relationships between binder content and estimated fatigue life per one percent optimum asphalt content are shown in Fig 33. These relationships indicate maximum economy occurs at binder contents between 5 and 10 percent for the Lubbock limestone mixtures and at approximately 5 percent for the Eagle Lake gravel mixtures. The latter is the same as the optimum binder content for maximum fatigue life; however, for the Lubbock limestone mixtures the optimum for maximum fatigue life was well defined at 5 percent rather than over a range between 5 and 10 percent.

Resilient Modulus of Elasticity

The relationships between asphalt content and the resilient modulus of elasticity at 0.5 N_f are shown in Fig 34 for Eagle Lake gravel mixtures and Fig 35 for Lubbock limestone mixtures. Both figures indicate that the optimum asphalt content for maximum resilient modulus is not well defined, with most of the relationships being flat. This behavior is consistent with the behavior reported by other investigators (Refs 1 and 26).

Nevertheless, to analyze the effects of binder content an attempt was made to pick an asphalt content which produced the maximum modulus. The resulting relationships between soil binder content and optimum asphalt content for maximum resilient modulus of elasticity for the loading cycle corresponding to 0.5 N_f are shown in Figs 36 and 37. The maximum resilient moduli of elasticity for Eagle Lake mixtures with 5 and 30 percent soil binder contents were about 2.5 times those for 0, 10, and 20 percent soil binder contents. Thus, either 5 or 30 percent may be chosen as the optimum soil binder content; however, the curve is not well defined. The optimum binder content for maximum resilient modulus of elasticity for the Lubbock limestone mixtures was 10 percent with 5.5 percent asphalt content.

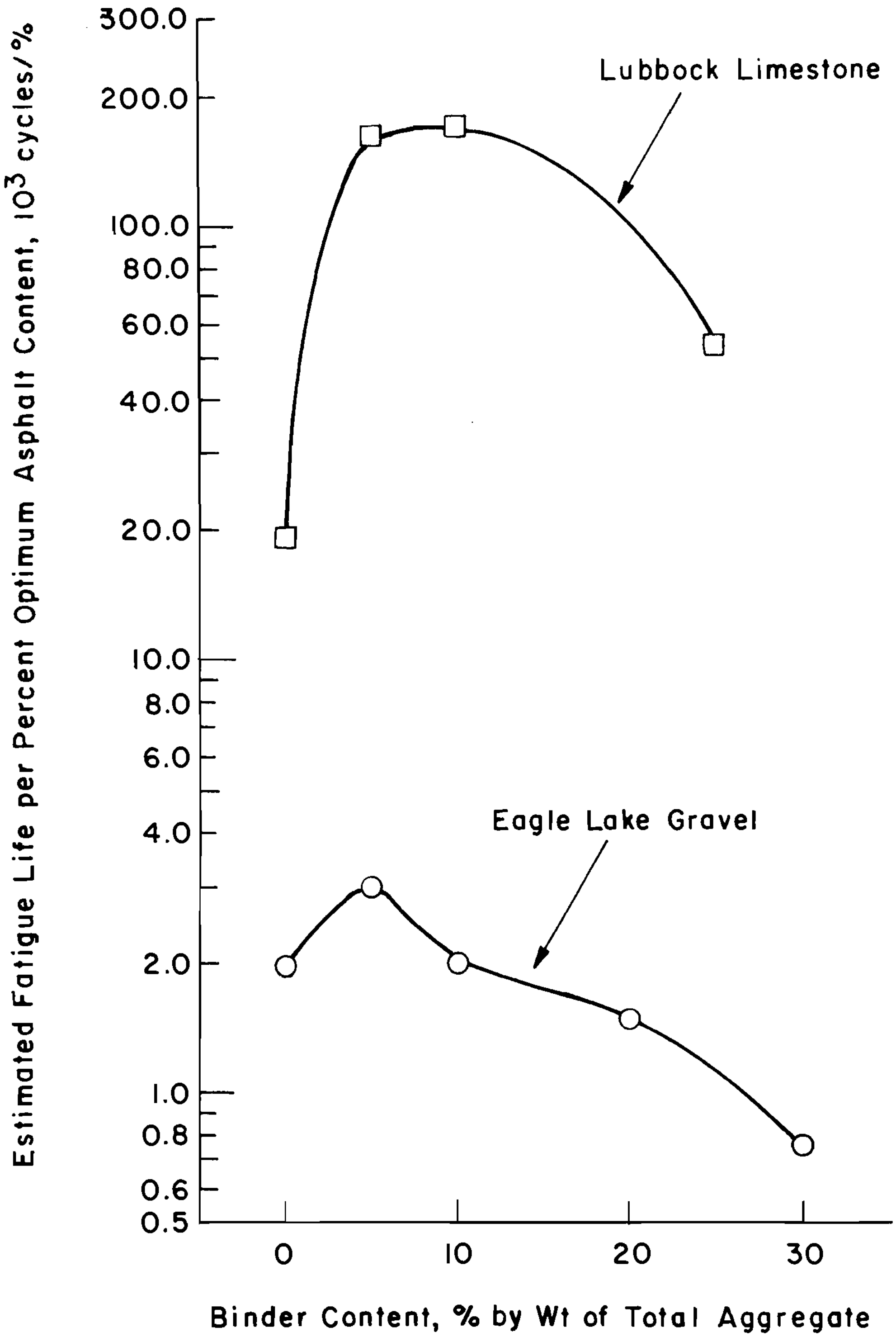


Fig 33. Relationships between binder content and the estimated fatigue life per unit percent of optimum asphalt content for Eagle Lake gravel and Lubbock limestone mixtures.

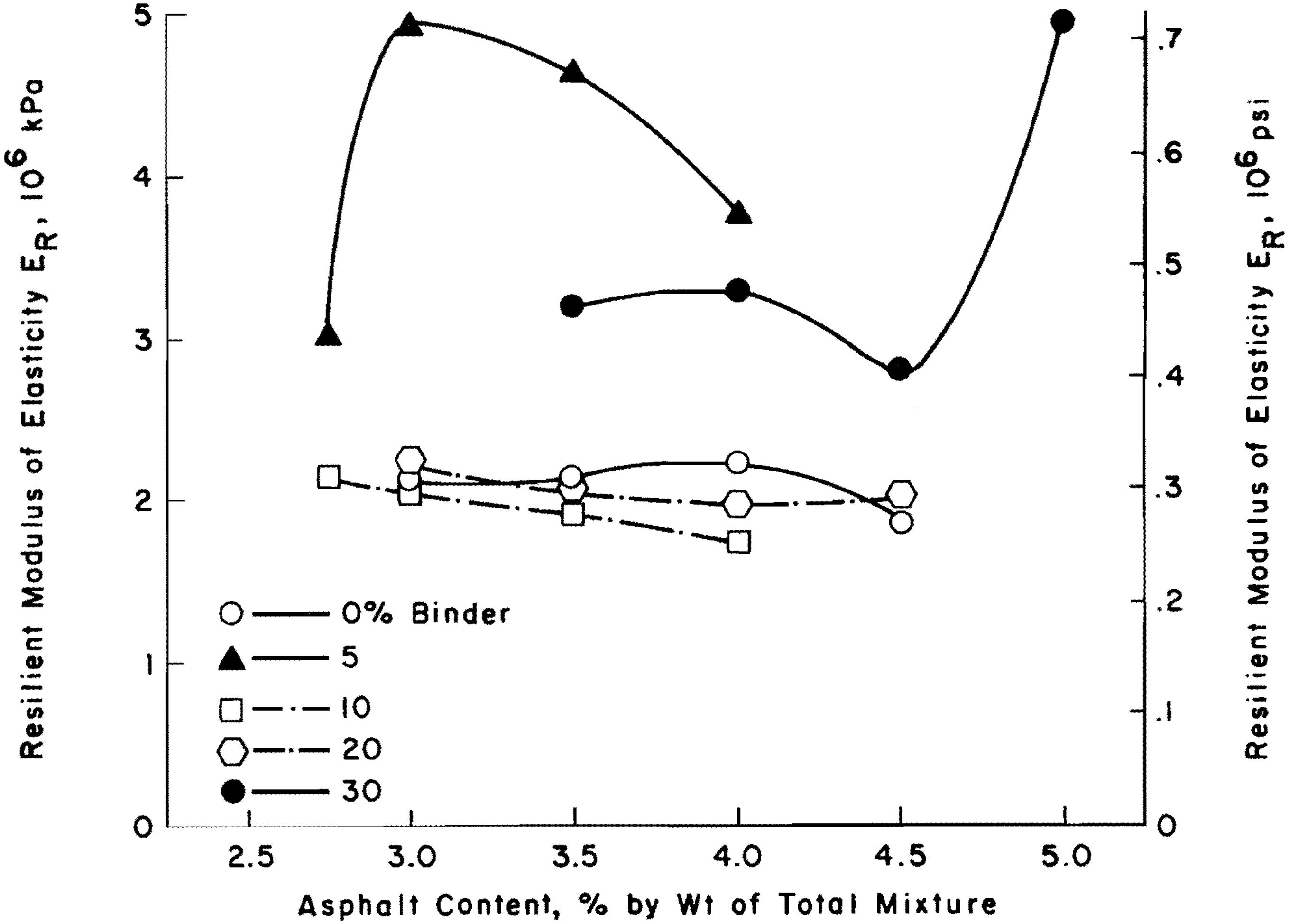


Fig 34. Relationships between asphalt content and resilient modulus of elasticity at $0.5N_f$ for Eagle Lake gravel mixtures.

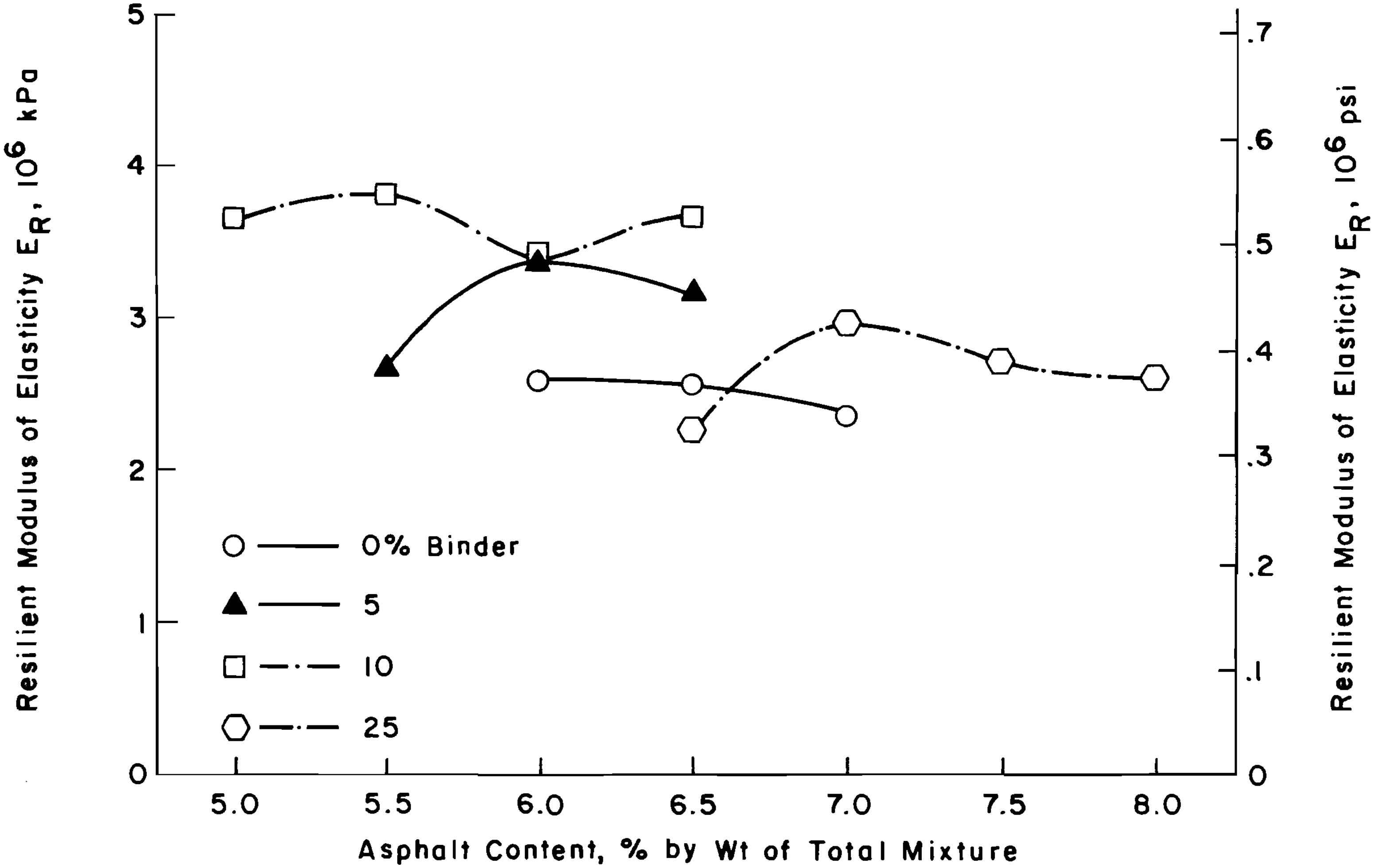


Fig 35. Relationships between asphalt content and resilient modulus of elasticity at $0.5N_f$ for Lubbock limestone mixtures.

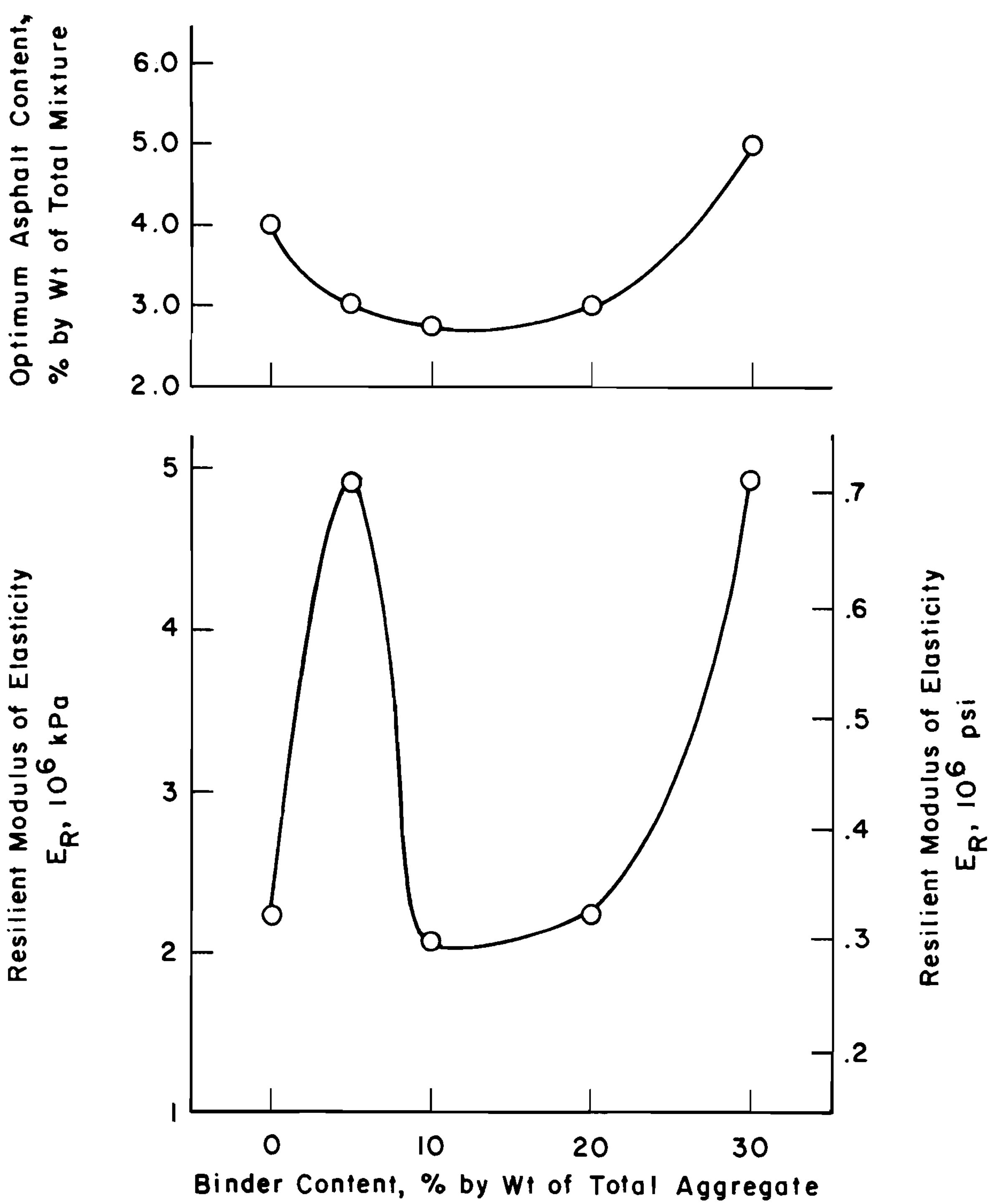


Fig 36. Relationships between soil binder content and both maximum resilient modulus of elasticity at $0.5N_f$ and the corresponding optimum asphalt content for Eagle Lake gravel mixtures.

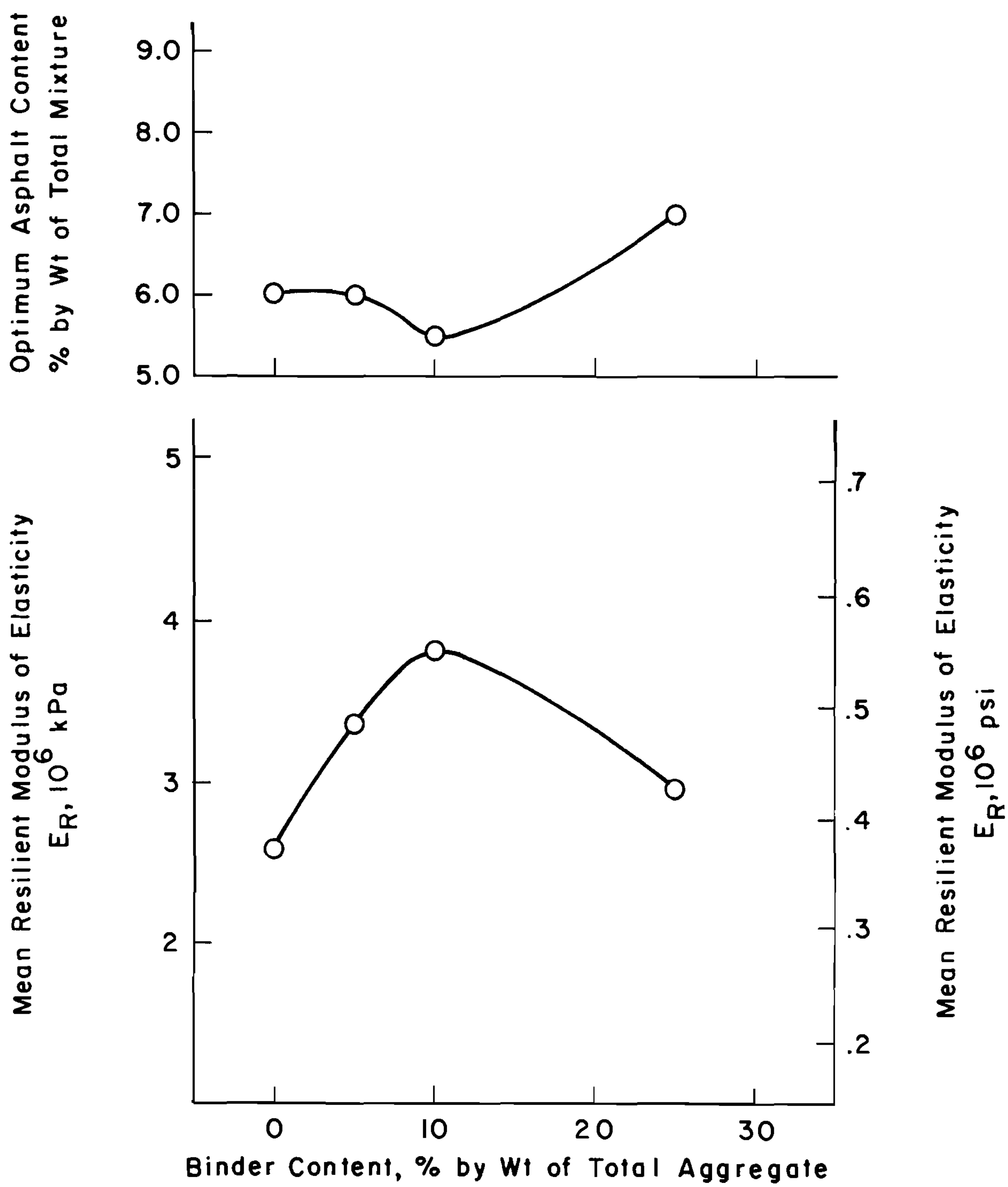


Fig 37. Relationships between soil binder content and both maximum resilient modulus of elasticity at $0.5N_f$ and the corresponding optimum asphalt content for Lubbock limestone mixtures.

Since it would appear that the actual asphalt content is not critical to resilient modulus of elasticity and since the actual values of modulus were relatively constant, the minimum values of asphalt content should be used.

Permanent Deformation

The effects of asphalt content on the rate of vertical permanent deformation for each binder content and stress level are shown in Figs 38 through 42 for Eagle Lake gravel and in Figs 43 through 46 for Lubbock limestone. As in previous studies (Refs 1 and 23), an optimum asphalt content for minimum rate of permanent deformation was found to occur. Values of rate of permanent deformation for each binder content for each of the two mixtures are presented in Appendix E.

For a constant stress level of 120 kPa (17.4 psi), the relationships between the soil binder content and both the optimum asphalt content and the corresponding minimum rate of permanent vertical deformation for Eagle Lake gravel mixtures for a stress level of 120 kPa (17.4 psi) are shown in Fig 47. Optimum asphalt contents ranged from 3.0 to 4.0 percent and the optimum binder content was 5 percent. For Lubbock limestone, the applied stress was different for the various mixtures. However, Fig 48 suggests that the optimum binder content for the lowest minimum rate of permanent deformation was approximately 10 percent for the Lubbock limestone mixtures.

It should be noted that the optimum asphalt content for minimum rate of permanent deformation appeared to be stress dependent. For the Eagle Lake mixtures the optimum asphalt content for minimum rate of permanent deformation was generally smaller for a higher stress level. For the Lubbock limestone mixtures this relationship was not well defined.

Moisture Damage

This study generally indicated that the optimum soil binder contents for maximum engineering properties were relatively low, in the range of 5 to 10 percent. In addition, these low binder contents required less asphalt and therefore improved the economy of the mixtures. However, the specimens tested were dry and had not been subjected to moisture. Thus, it was necessary to evaluate the effects of water on the engineering properties of the two materials as discussed in Chapter 2. A series of specimens for each aggregate type at the optimum asphalt content for the maximum ultimate tensile strength were

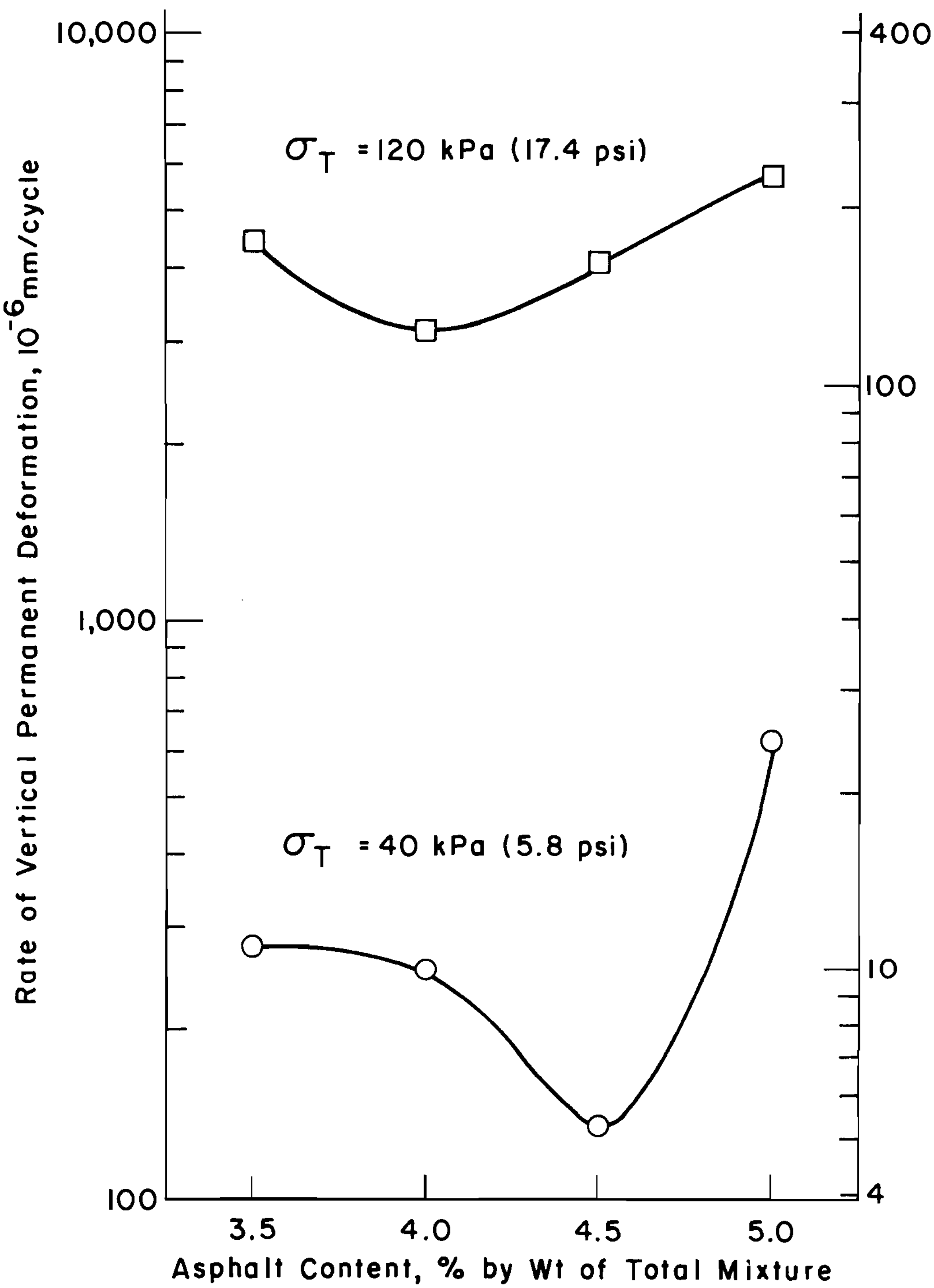


Fig 38. Relationships between asphalt content and rate of permanent deformation for Eagle Lake gravel mixtures with 30 percent binder.

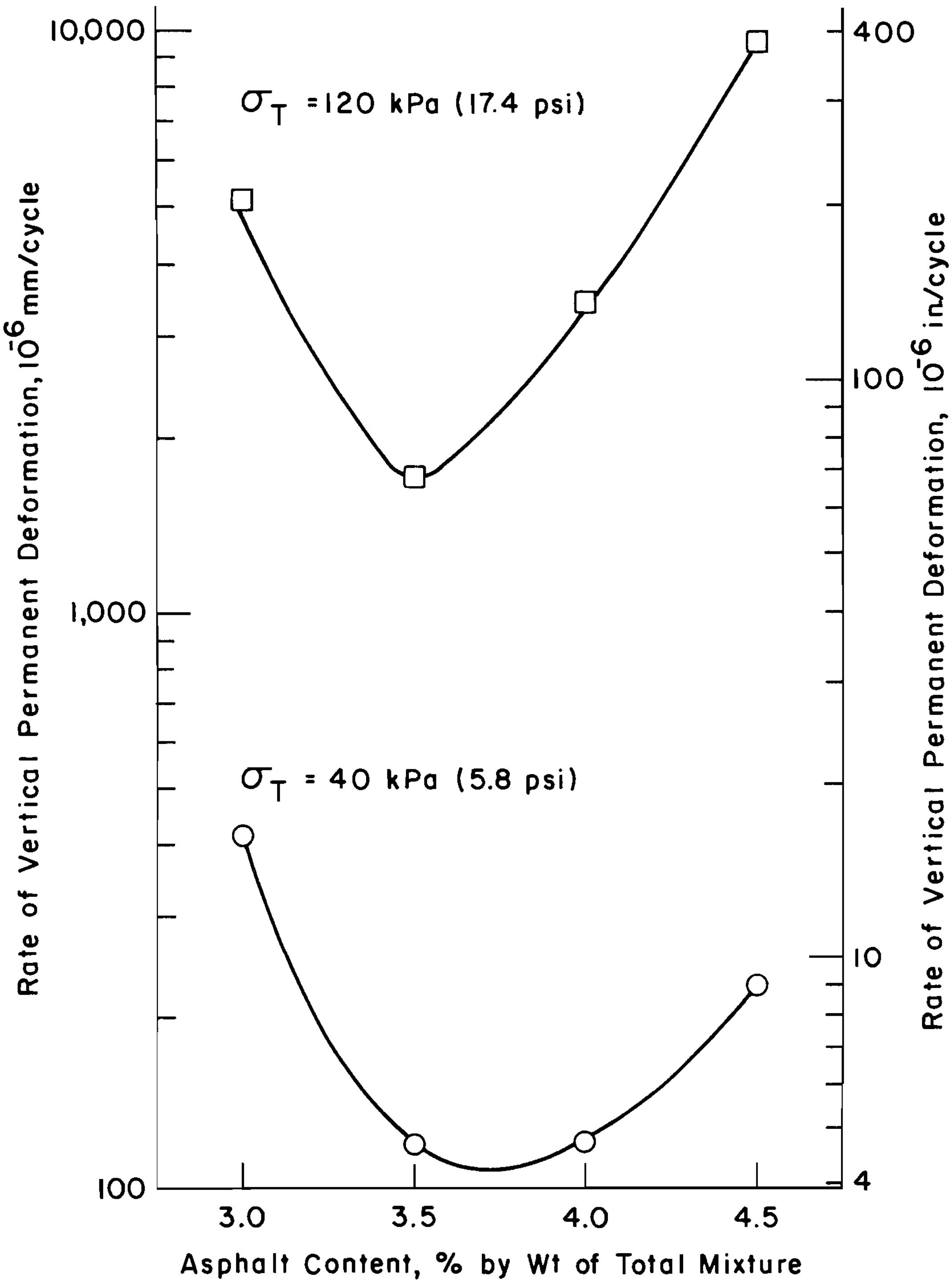


Fig 39. Relationships between asphalt content and rate of permanent deformation for Eagle Lake gravel mixtures with 20 percent binder.

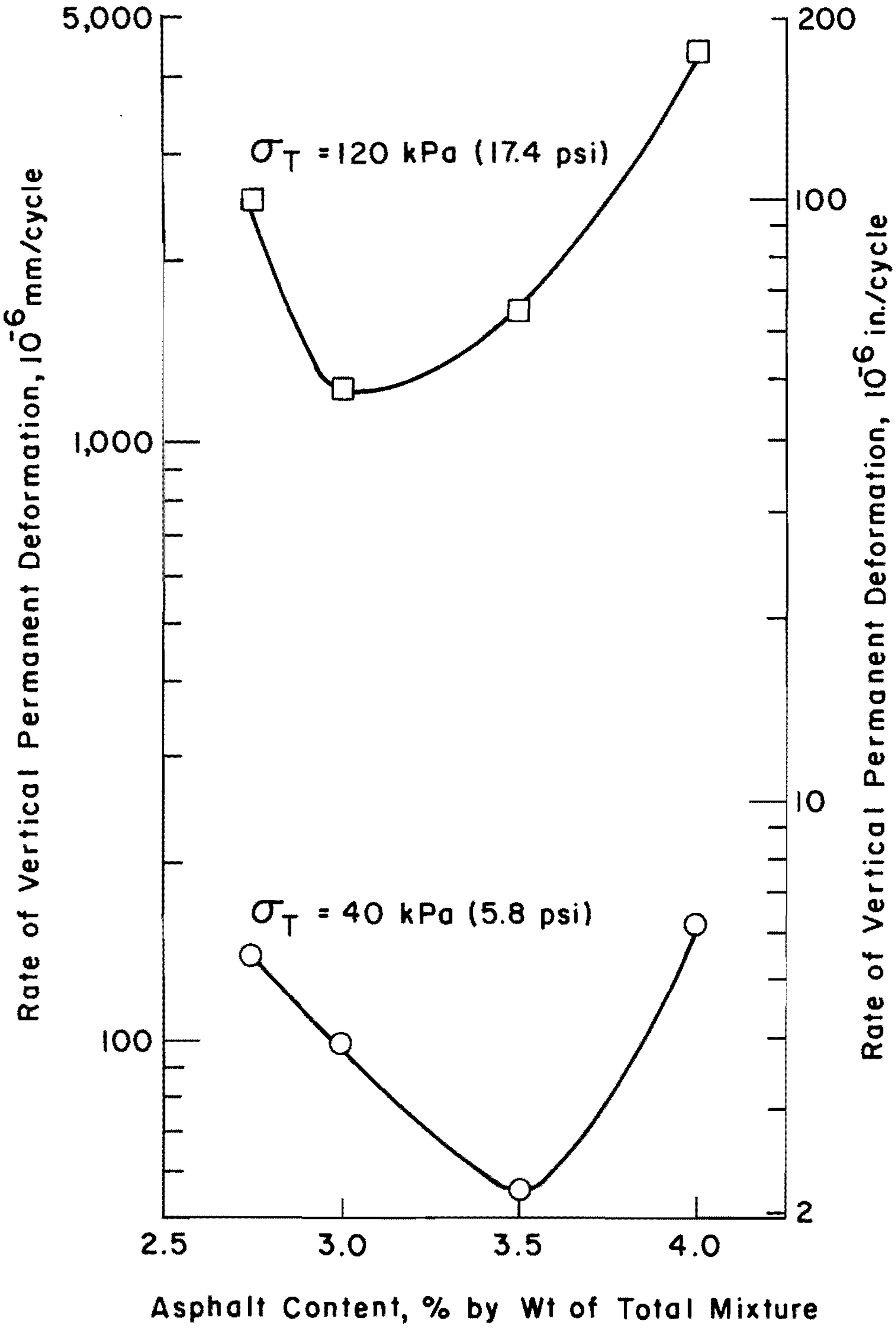


Fig 40. Relationships between asphalt content and rate of permanent deformation for Eagle Lake gravel mixtures with 10 percent binder.

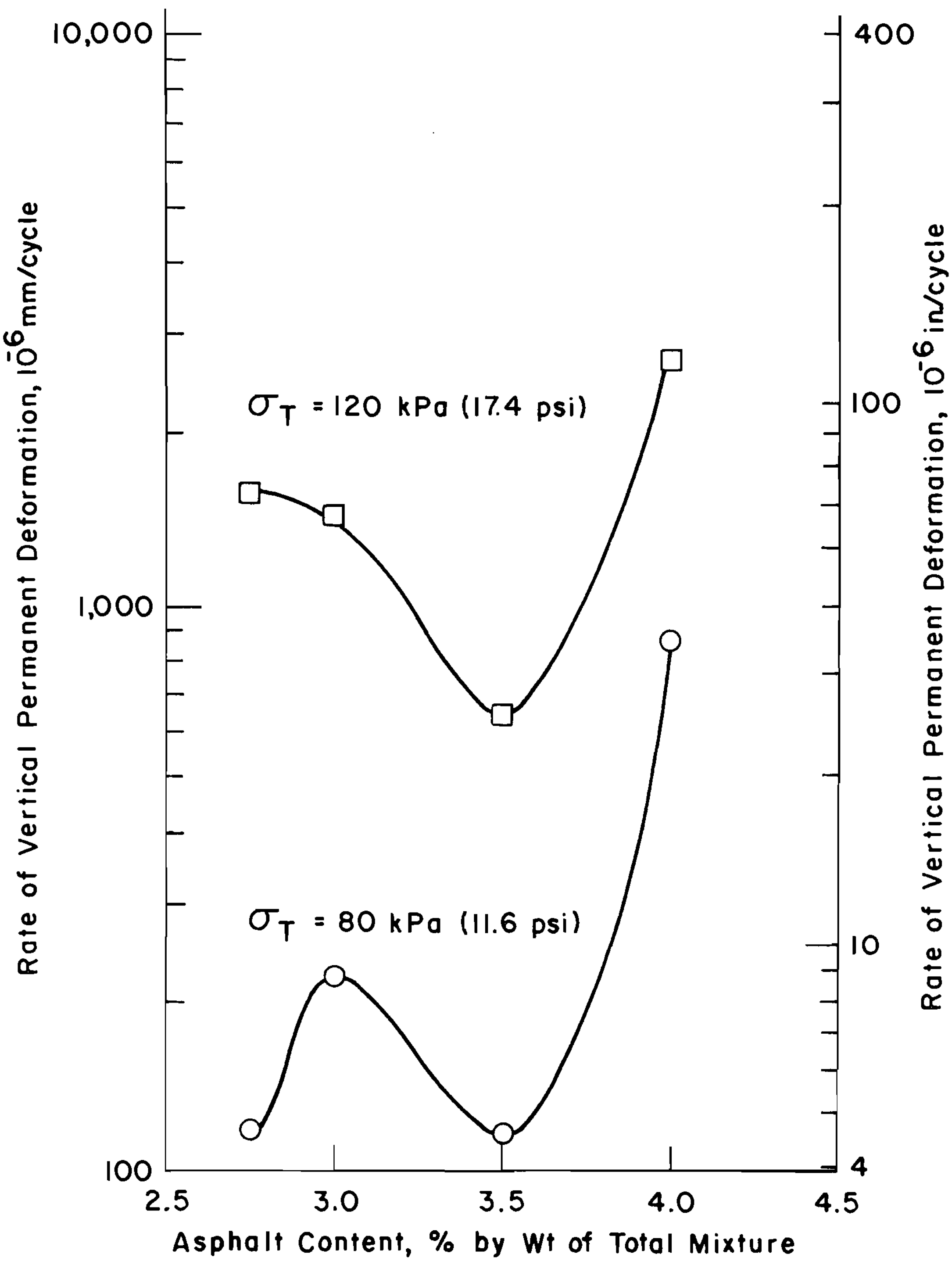


Fig 41. Relationships between asphalt content and rate of permanent deformation for Eagle Lake gravel mixtures with 5 percent binder.

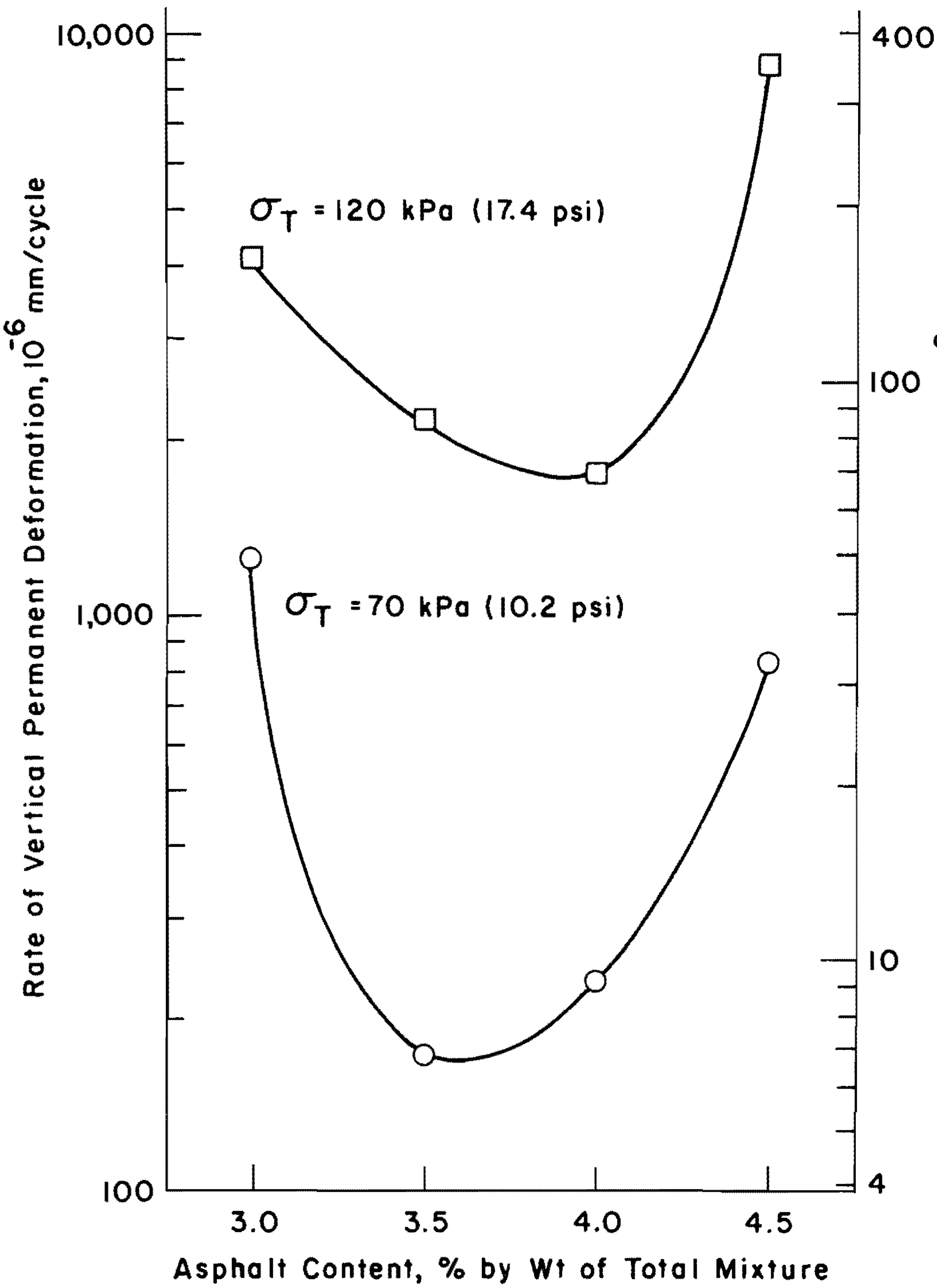


Fig 42. Relationships between asphalt content and rate of permanent deformation for Eagle Lake gravel mixtures with 0 percent binder.

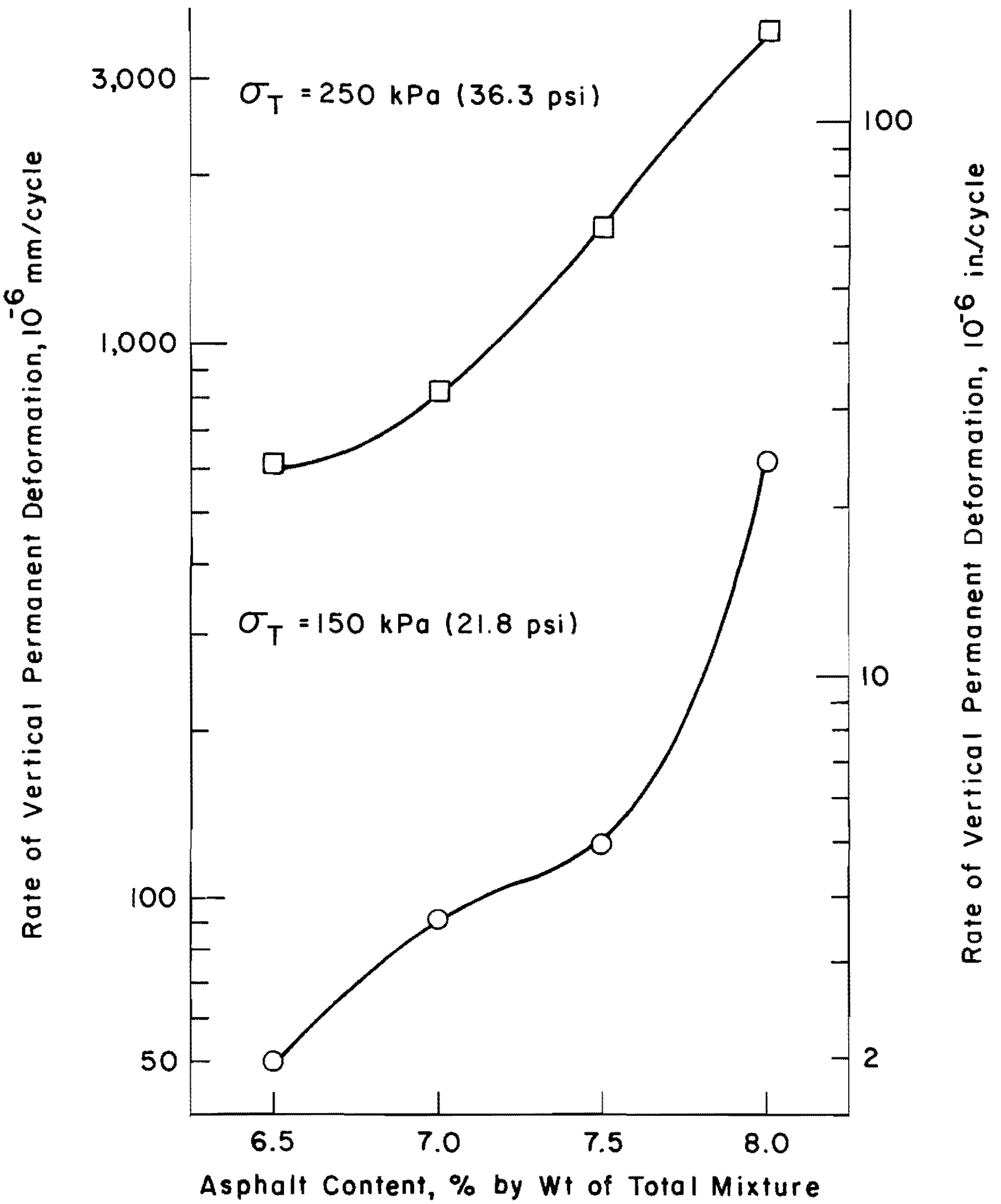


Fig 43. Relationships between asphalt content and rate of permanent deformation for Lubbock limestone mixtures with 25 percent binder.

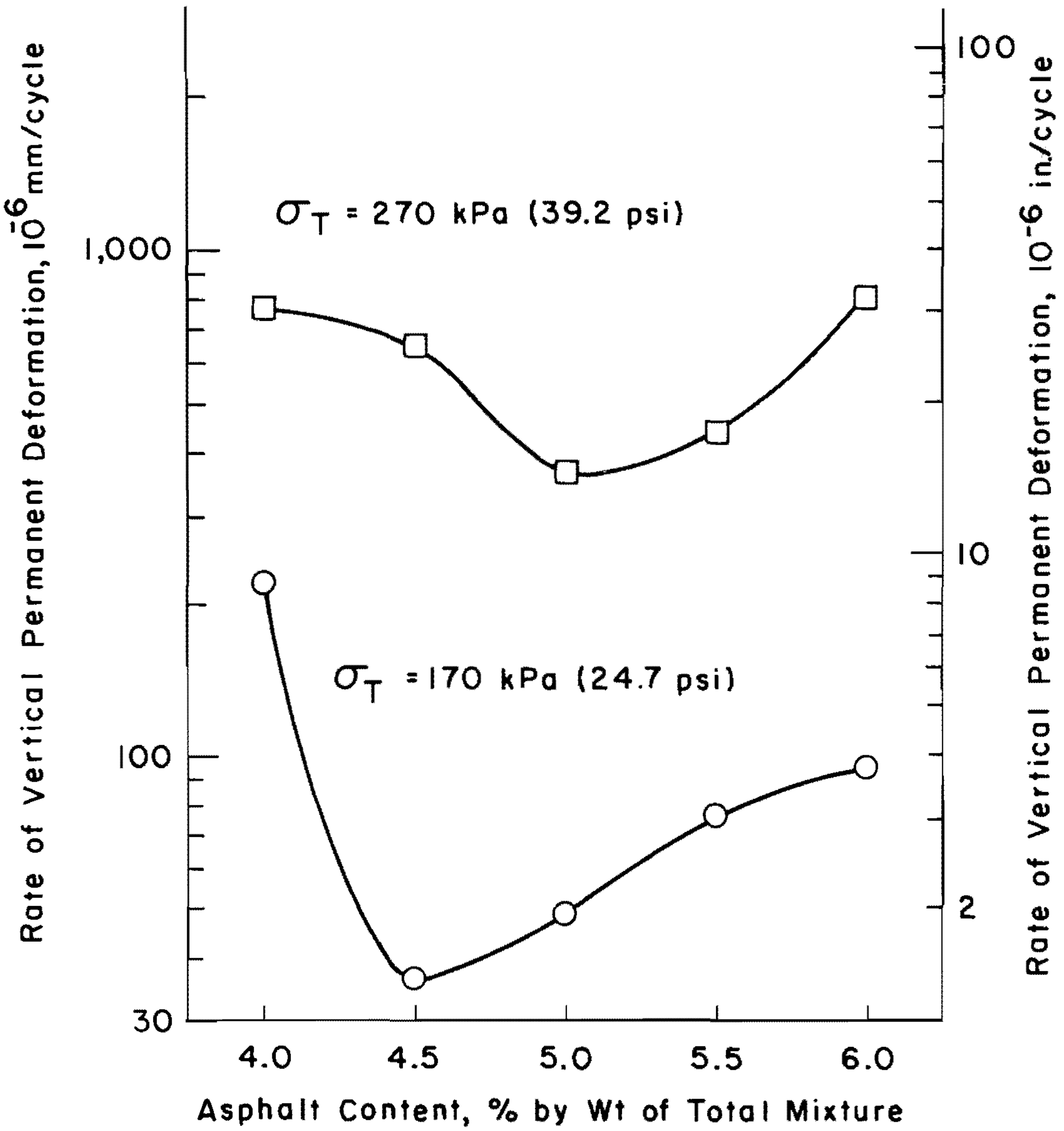


Fig 44. Relationships between asphalt content and rate of permanent deformation for Lubbock limestone mixtures with 10 percent binder.

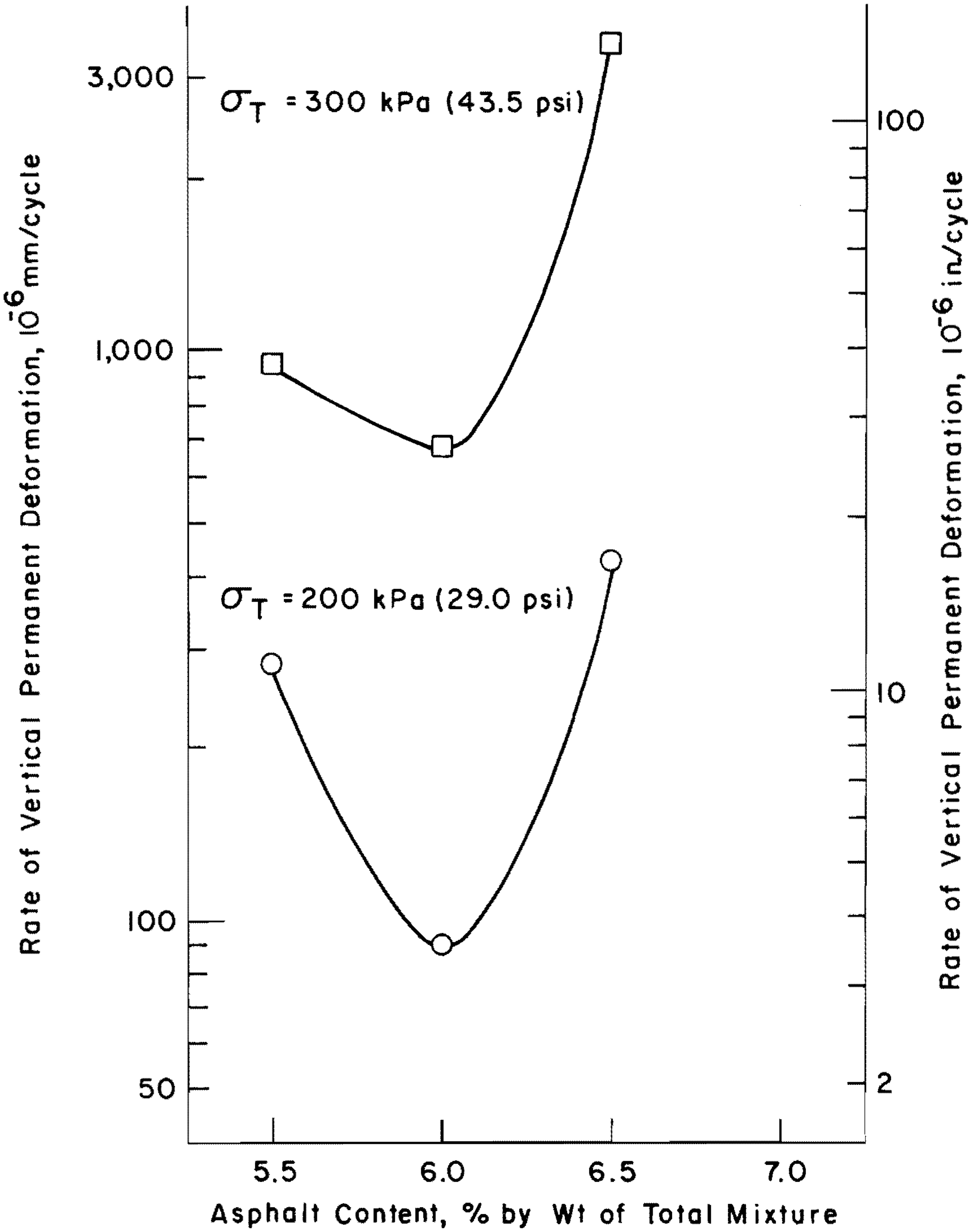


Fig 45. Relationships between asphalt content and rate of permanent deformation for Lubbock limestone mixtures with 5 percent binder.

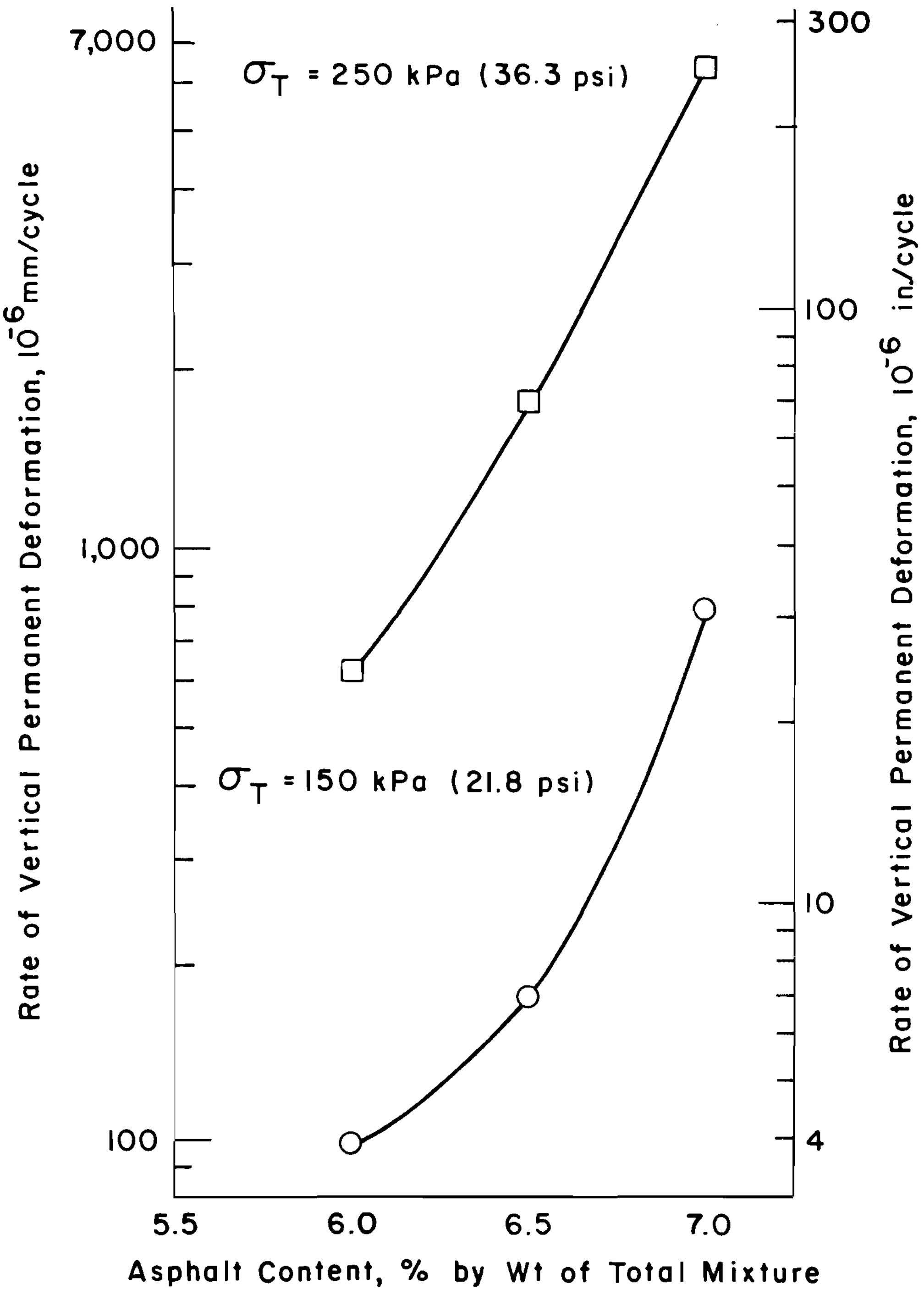


Fig 46. Relationships between asphalt content and rate of permanent deformation for Lubbock limestone mixtures with 0 percent mixtures.

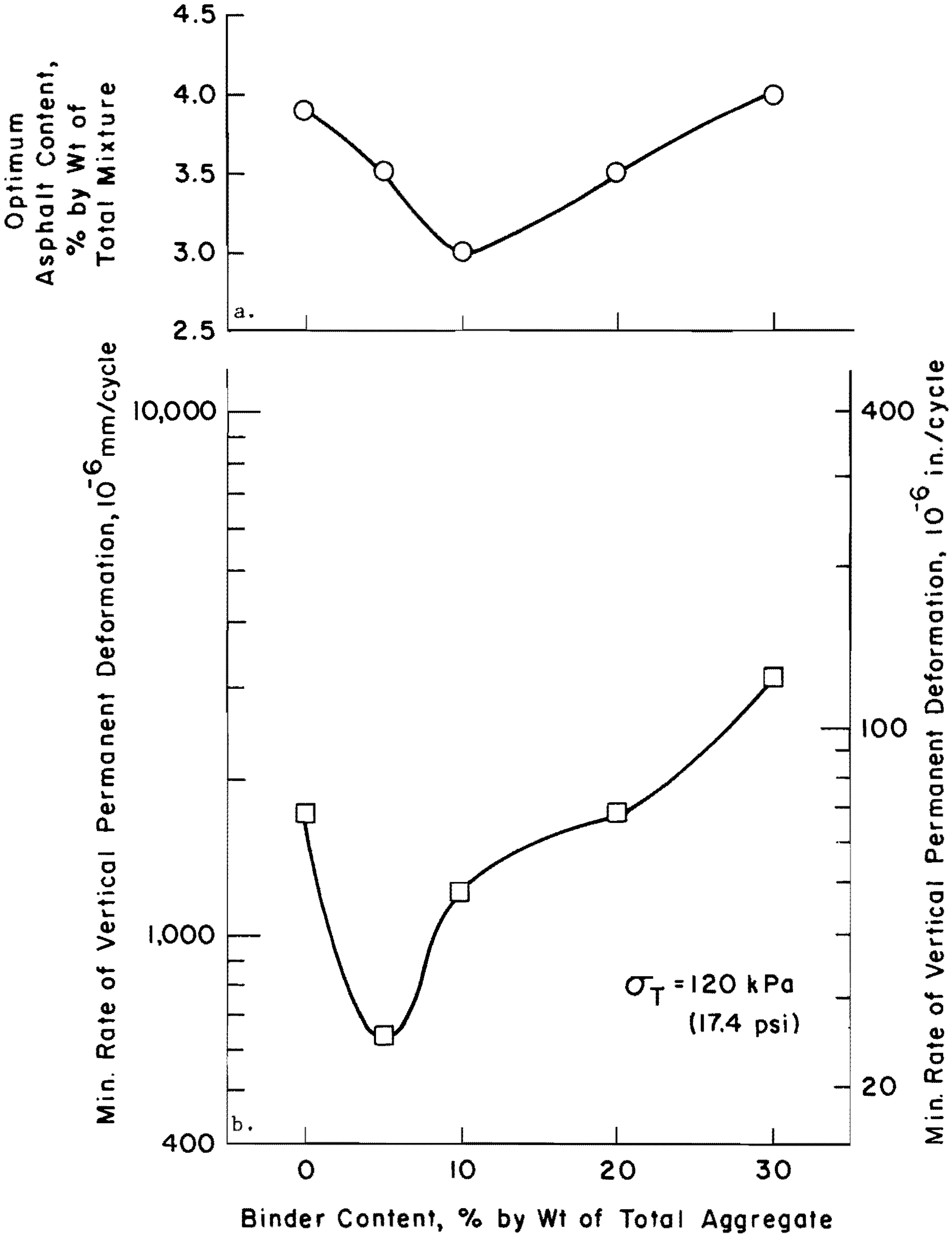


Fig 47. Relationships between soil binder content and both optimum asphalt content and the corresponding minimum rate of permanent deformation for Eagle Lake gravel mixtures.

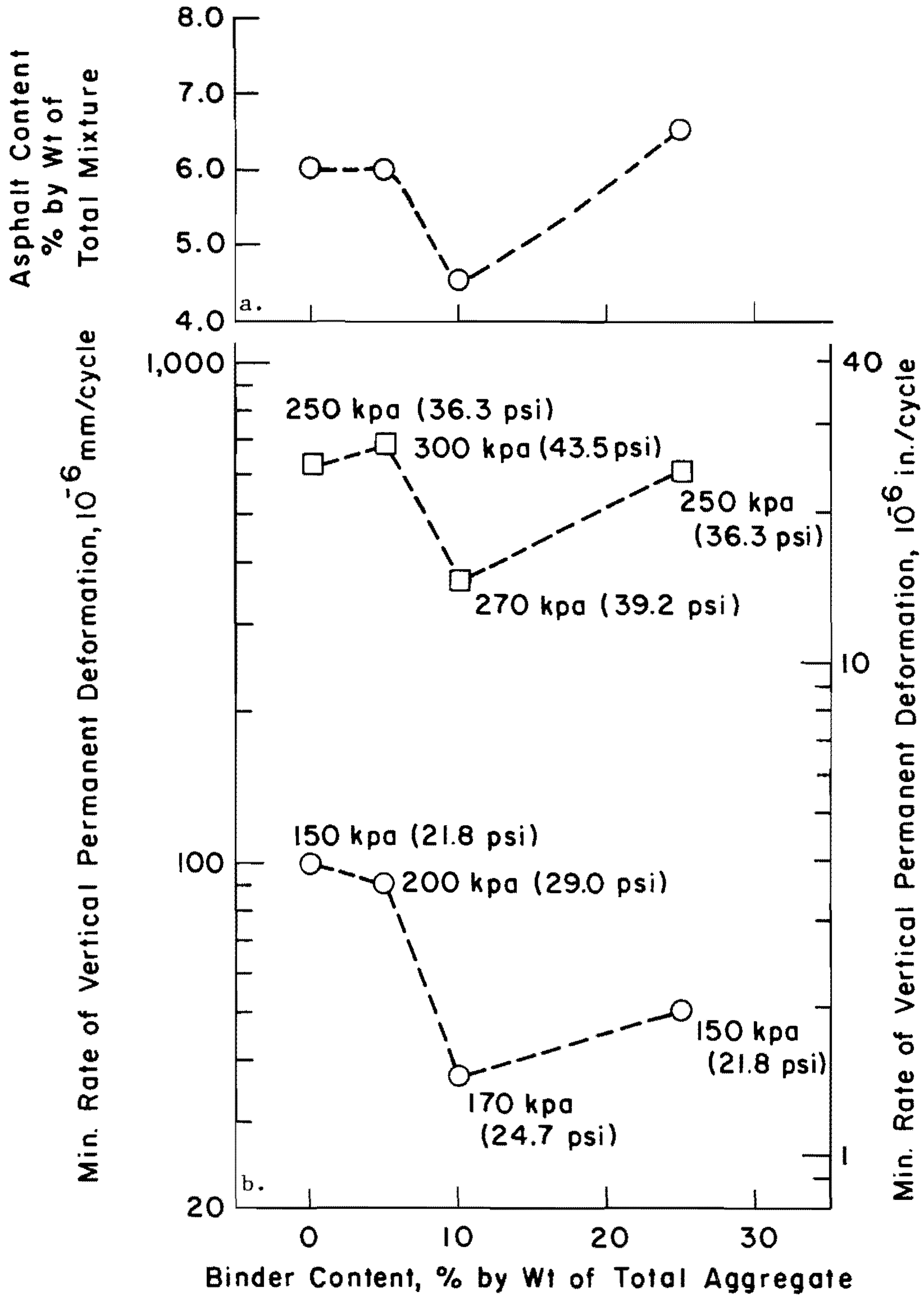


Fig 48. Relationship between soil binder content and both optimum asphalt content and the corresponding minimum rate of vertical permanent deformation at different stress levels for Lubbock limestone mixtures.

subjected to pressure wetting and then were tested to obtain static indirect tensile results and to obtain the resilient modulus of elasticity.

Test results are shown in Figs 49 through 56. The relationships between binder content and the asphalt content, total air voids, water content after pressure wetting, and densities of the specimens are shown in Figs 49 and 50 for Eagle Lake gravel mixtures and in Figs 51 and 52 for Lubbock limestone mixtures. Total air voids and densities of tested specimens were not exactly the same as those obtained from the specimens used to establish the laboratory AVR relationships, but the values were close. The asphalt contents of tested specimens were lower than the optimum asphalt contents for the maximum densities and thus the corresponding densities were less than the maximum densities and the air void contents were higher. As shown in Figs 49 and 51, water contents after pressure wetting were proportional to the total air voids, i.e., the higher the total air voids, the higher the water contents.

There was a definite effect of moisture on the ultimate tensile strength and the static modulus of elasticity (Figs 53 and 55). A strength loss of about 250 kPa (36 psi) occurred for Eagle Lake gravel mixtures with 5 percent soil binder and of about 500 kPa (72 psi) for mixtures with 30 percent soil binder. However, pressure wetting did not produce a loss of tensile strength for mixtures with 0, 10, and 20 percent soil binder. For the Lubbock limestone mixtures the losses were more consistent, varying from 750 kPa (110 psi) to 400 kPa (58 psi). The effect of pressure wetting on static modulus of elasticity was more significant (Figs 53a and 55a). Losses in modulus for the Eagle Lake mixtures ranged from 100,000 kPa (14,500 psi) to slightly less than 1,000,000 kPa (145,000 psi). Similarly, for the Lubbock limestone the losses ranged from about 400,000 kPa (58,000 psi) to 1,000,000 kPa (145,000 psi). No consistent or explainable relationships were observed for the resilient modulus of elasticity (Figs 54b and 56b). In most cases the pressure wetted specimens exhibited higher moduli than the dry specimens. This was especially true for the Lubbock limestone mixtures.

A comparison of the density relationships for tested specimens (Figs 50b and 52b) with the curves of the ultimate tensile strength and the static modulus of elasticity after pressure wetting (Figs 53 and 55) indicates that the shapes are similar.

Thus, it would appear that moisture damage was dependent on the density of the mixture, or air void content. It was found that the highest density

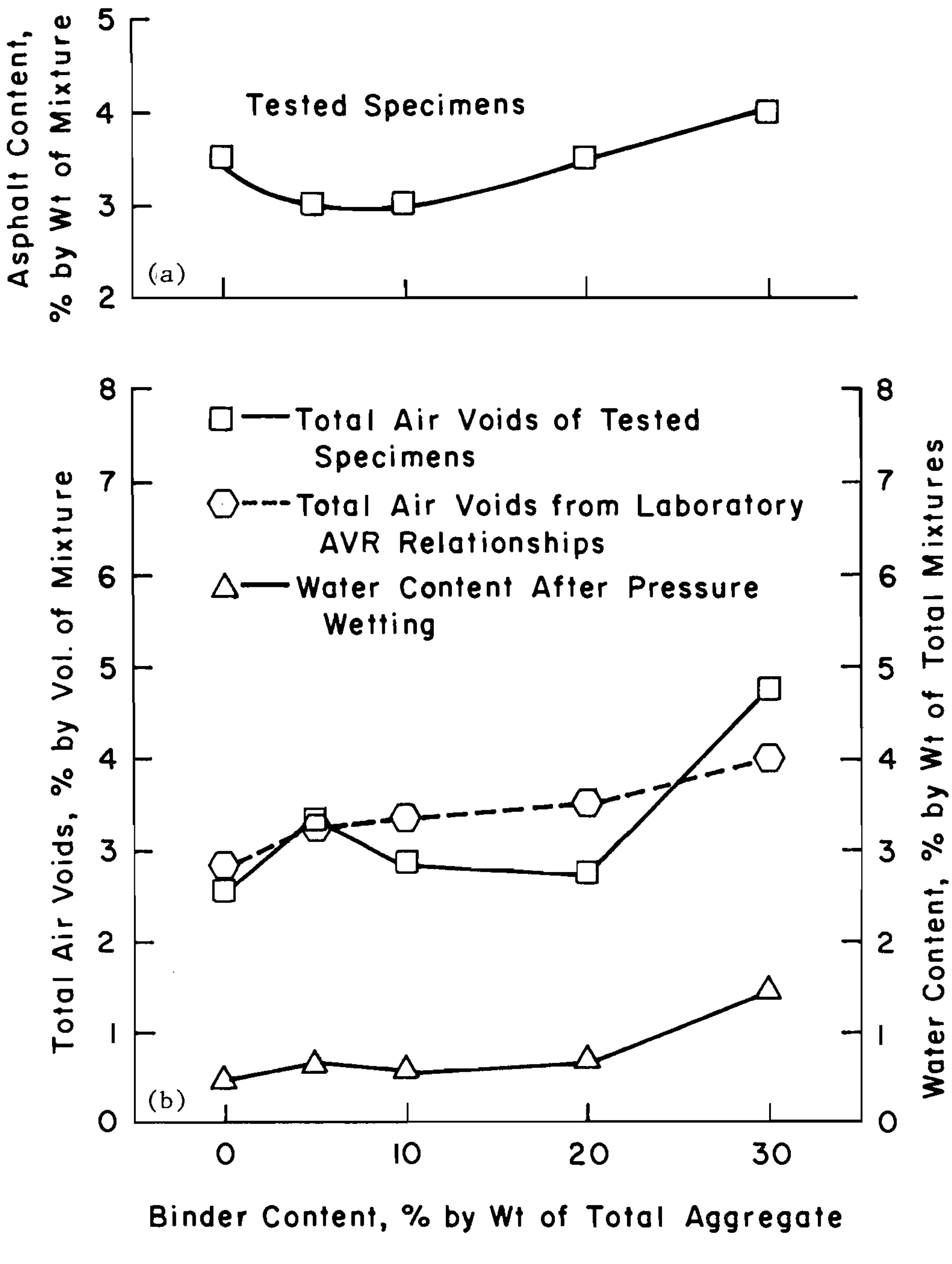


Fig 49. Relationships between binder content and asphalt content, total air voids, and water content of specimens containing Eagle Lake gravel.

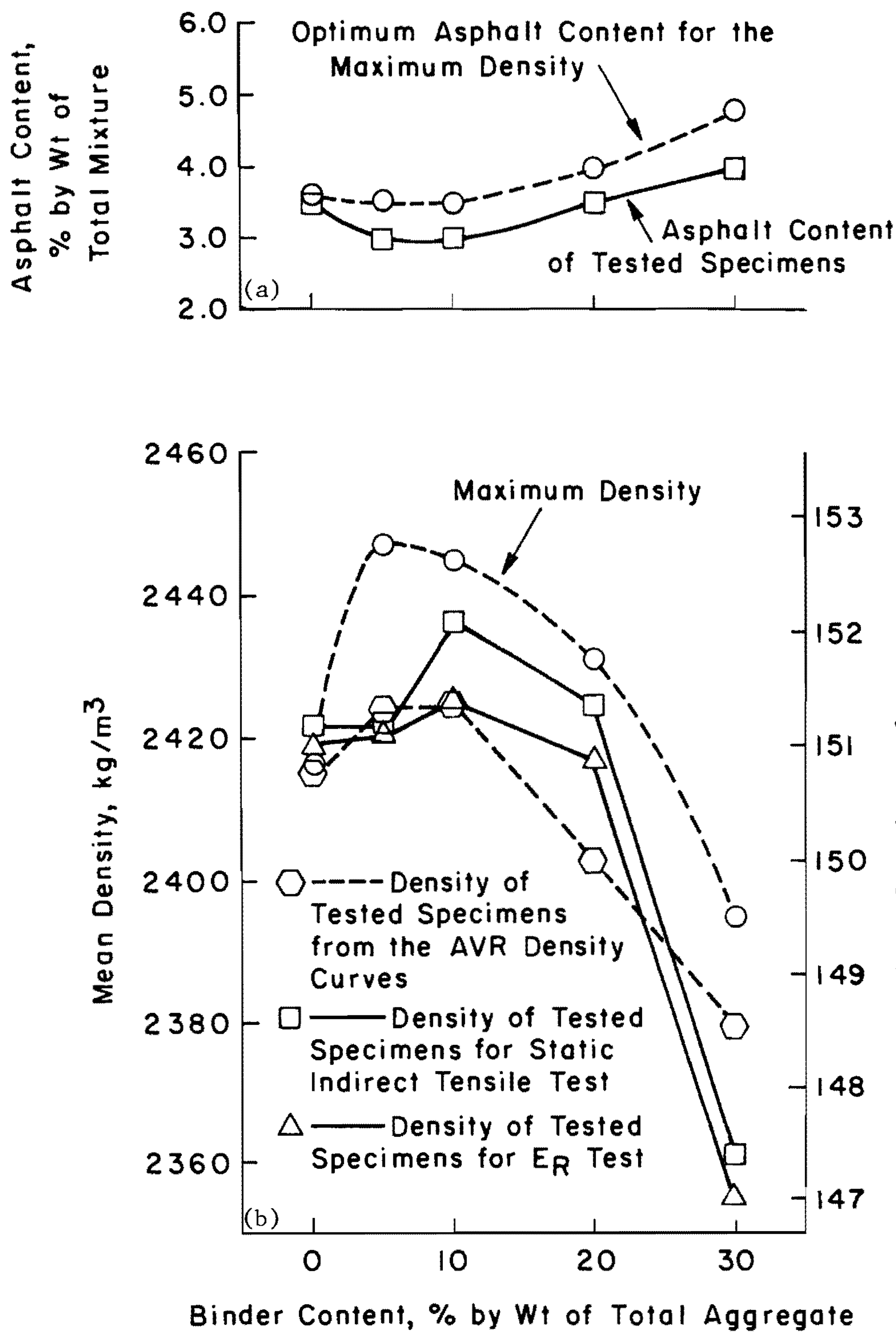


Fig 50. Relationships between binder content and the asphalt contents and densities of tested specimens of Eagle Lake gravel mixtures.

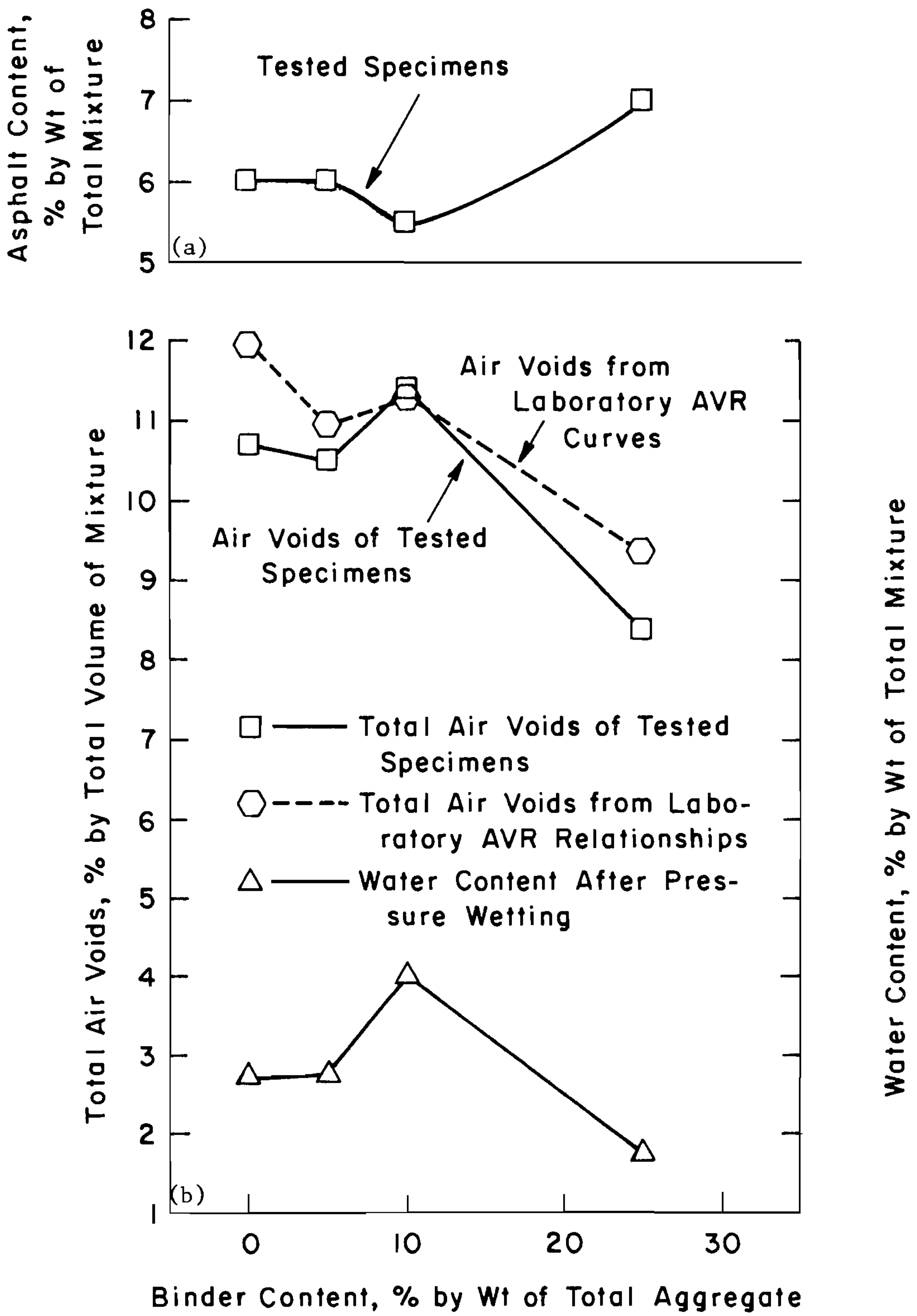


Fig 51. Relationships between binder content and asphalt content, total air voids, water content of specimens containing Lubbock limestone.

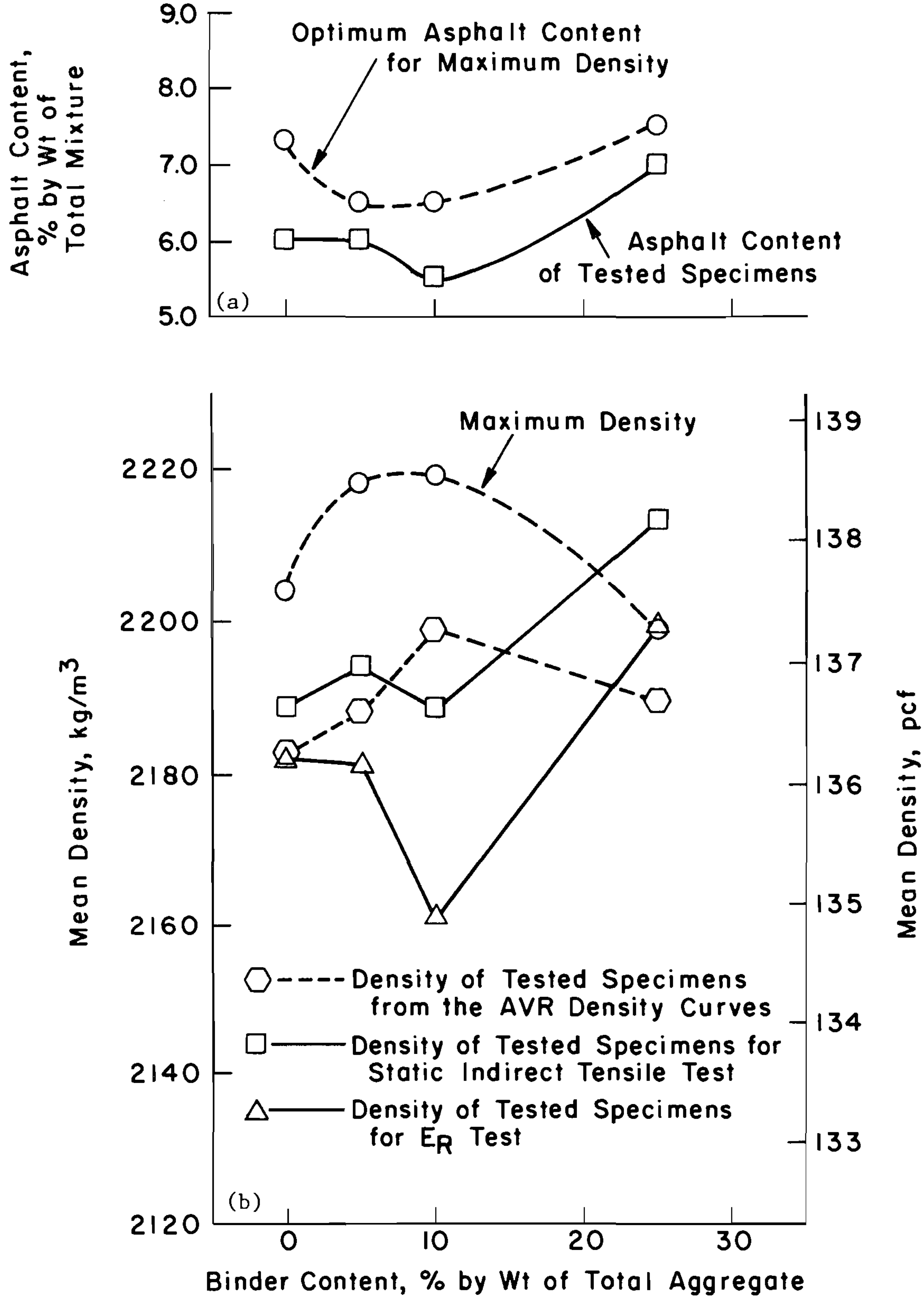


Fig 52. Relationships between binder content and the asphalt contents and densities of tested specimens of Lubbock limestone mixtures.

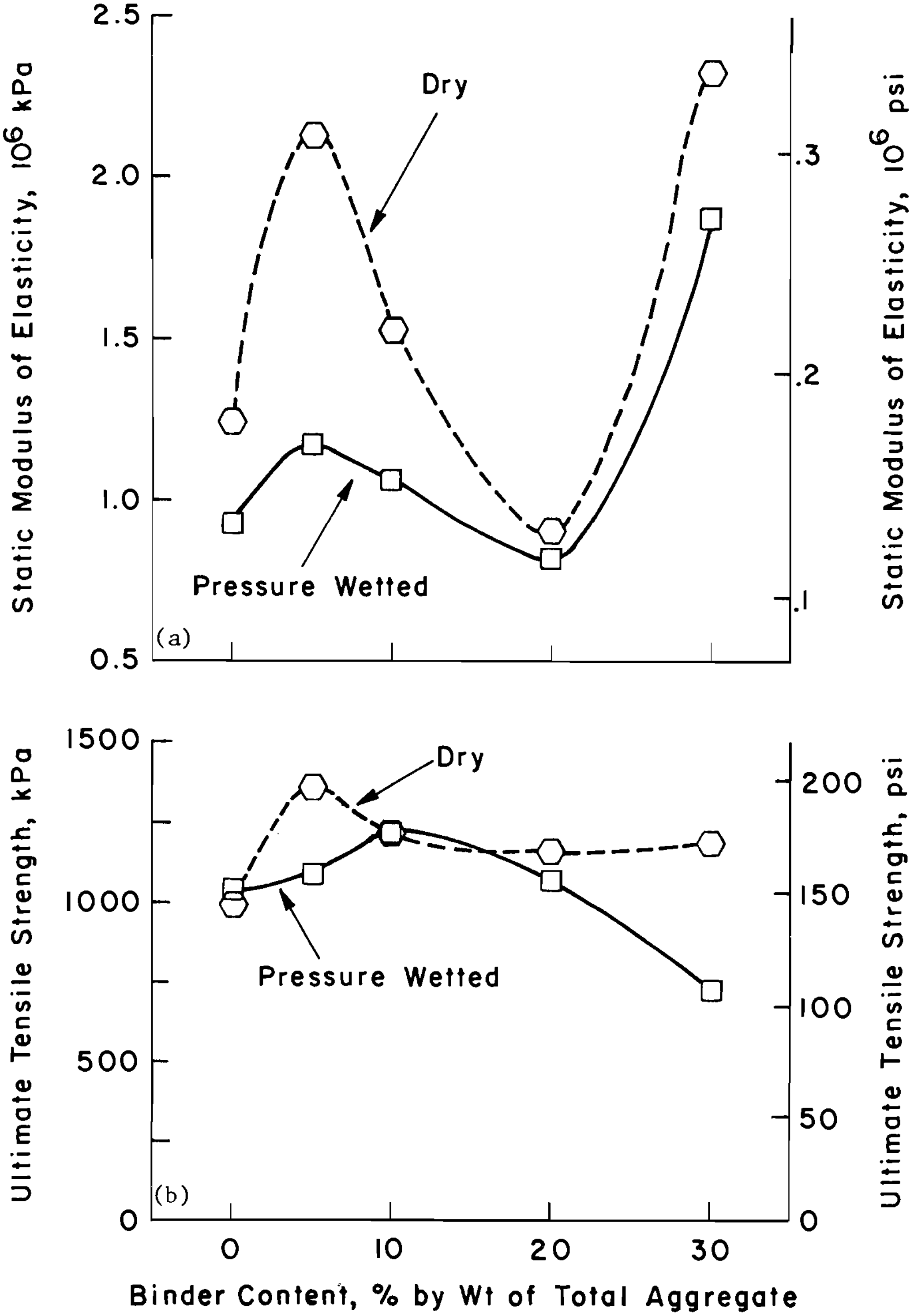


Fig 53. Relationships between binder content and moisture content on the ultimate tesnile strength and static modulus of elasticity for Eagle Lake gravel mixtures.

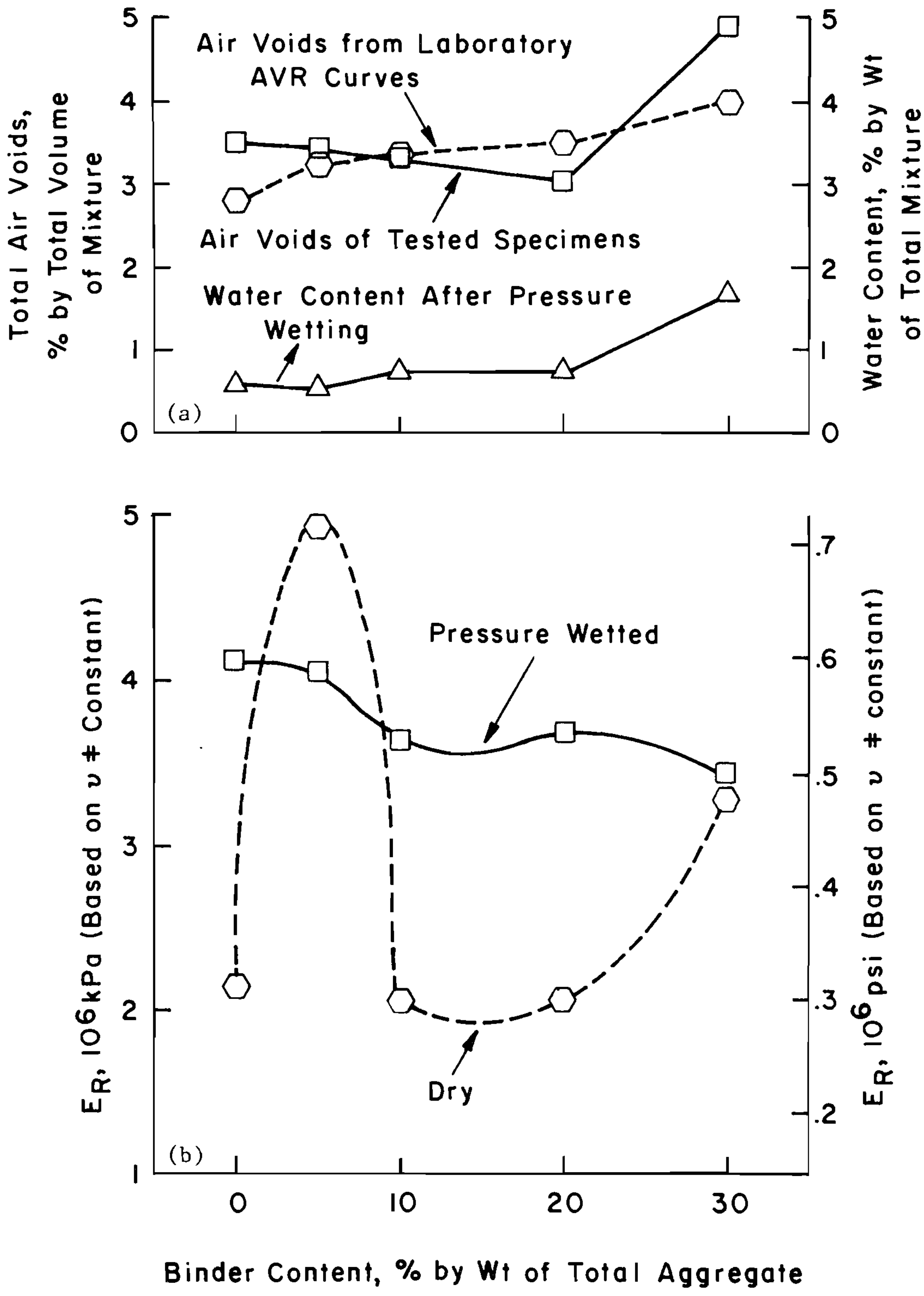


Fig 54. Relationships between binder contents and air voids, water content, and resilient modulus of elasticity for Eagle Lake gravel mixtures.

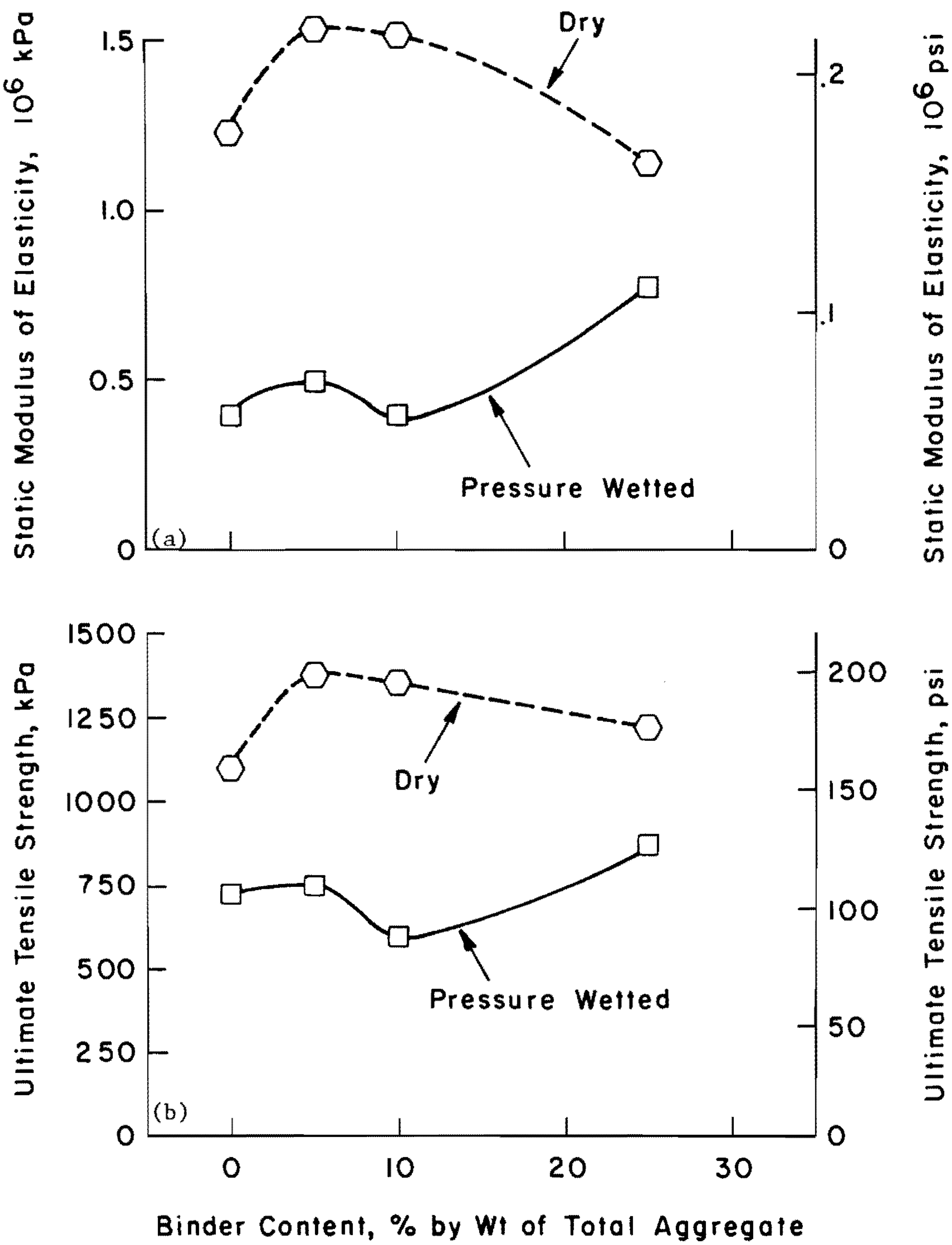


Fig 55. Relationships between binder content and moisture content on the ultimate tensile strength and the static modulus of elasticity for Lubbock limestone mixtures.

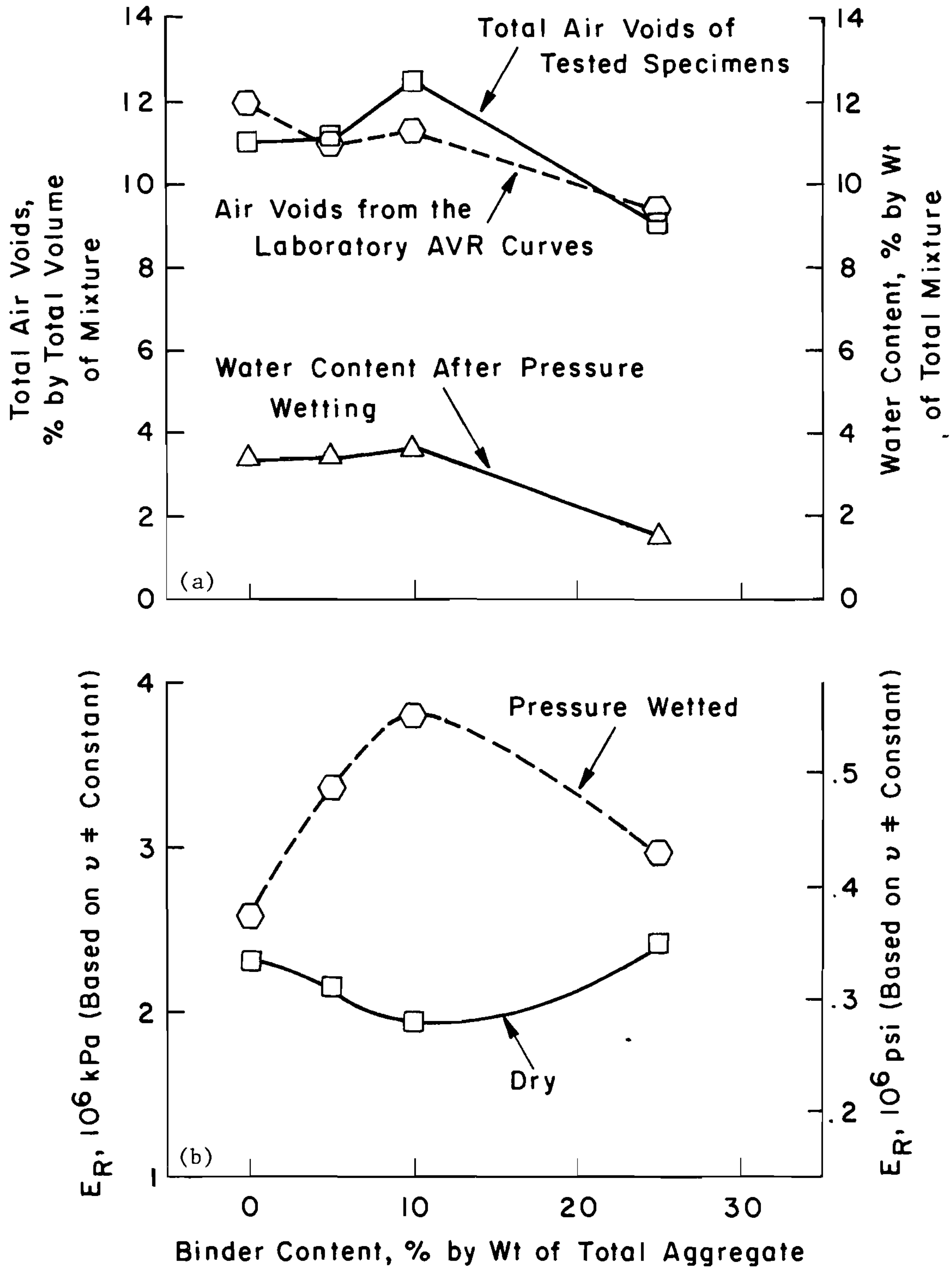


Fig 56. Relationships between content and air voids, and water content, and resilient modulus of elasticity for Lubbock limestone mixtures.

for Eagle Lake gravel mixtures was achieved at 5 percent soil binder content and for Lubbock limestone mixtures at 10 percent soil binder content. This would suggest that as long as the mixture has adequate density substantial damage will not occur; however, it must be kept in mind that this was a very limited study concerning the effect of moisture.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations based on the findings of this investigation are summarized below.

CONCLUSIONS

General

(1) The laboratory AVR design optimum asphalt contents ranged from 3.5 percent to 4.5 percent for Eagle Lake gravel mixtures and from 6.6 percent to 7.3 percent for Lubbock limestone mixtures. Total air voids were affected by both the soil binder content and the asphalt content. With proper asphalt content, the minimum total air voids occurred at soil binder contents between 7 and 10 percent for Eagle Lake gravel mixtures and at 10 percent soil binder content for Lubbock limestone mixtures. The corresponding total air voids for Lubbock limestone mixtures with the laboratory AVR design optimum asphalt content were from 8.6 percent to 9.0 percent, which were much higher than those for Eagle Lake gravel mixtures (1.6 percent to 2.7 percent).

(2) An optimum binder content for the maximum AVR density existed for the two materials. The Eagle Lake gravel mixture with 5 percent soil binder content had the lowest optimum asphalt content (3.5 percent) and the highest density (2,447 kg/m^3) while the Lubbock limestone mixture with 10 percent soil binder content had the lowest optimum asphalt content (6.5 percent) and the highest density (2,219 kg/m^3).

(3) There was a tendency for the optimum asphalt content to decrease as the soil binder content decreased; however, when the mixture contained little or no soil binder the optimum asphalt content increased.

Static Characteristics

(1) Both Eagle Lake gravel and Lubbock limestone mixtures exhibited essentially equal maximum static tensile strengths; however, the asphalt content required for the Lubbock limestone mixtures (6 percent) was much higher than that for Eagle Lake gravel mixtures (3 percent).

(2) Within the limits of this study, no definite relationship could be found between static modulus of elasticity and soil binder content for Eagle Lake gravel mixtures; however, an optimum soil binder content (10 percent) for maximum static modulus of elasticity existed for Lubbock limestone mixtures.

Repeated-Load Characteristics

(1) A definite optimum binder content for maximum fatigue life existed for both the Eagle Lake gravel and the Lubbock limestone mixtures. At the same stress level (100 kPa), the fatigue life of Lubbock limestone mixtures was much higher than that of Eagle Lake gravel mixtures, i.e., the maximum fatigue life of Lubbock limestone mixtures at 5 percent binder content was about 110 times that of Eagle Lake gravel mixtures.

(2) For resilient modulus of elasticity and static modulus of elasticity no well defined optimum soil binder content existed for the Eagle Lake gravel mixture; however, an optimum soil binder content (10 percent) was found for the Lubbock limestone mixture.

(3) For permanent deformation, an optimum soil binder content (5 percent) was found for the Eagle Lake gravel mixture. Although the data were insufficient for the Lubbock limestone mixtures, the general tendency indicated that an optimum soil binder content for minimum permanent deformation per cycle existed somewhere around 5 percent.

Moisture Damage

(1) The moisture damage for the Lubbock limestone mixture was more severe than for the Eagle Lake gravel mixture.

(2) The moisture damage appeared to be directly related to the total air voids of the mixture, i.e., the damage due to water was greater for mixtures with higher total air voids.

Optimum Asphalt Content

(1) Optimum asphalt contents were found to occur for the following material properties:
   (a) AVR density,
   (b) tensile strength,
   (c) static modulus of elasticity,
   (d) fatigue life, and
   (e) permanent deformation.

   No well-defined optimum occurred for the resilient modulus of elasticity.

(2) The optimum asphalt content for mixtures with higher soil binder content was generally larger than the optimum for mixtures with lower soil binder content.

(3) In general, the lowest optimum asphalt content occurred at 5 percent soil binder for the Eagle Lake gravel mixture and at 10 percent soil binder for the Lubbock limestone mixture.

Optimum Soil Binder Content

(1) For the Eagle Lake gravel mixture, the optimum soil binder content was 5 percent for AVR density, tensile strength, fatigue life, and permanent deformation.

(2) For the Lubbock limestone mixture, the optimum soil binder content ranged from 5 to 10 percent for the various engineering properties.

RECOMMENDATIONS

(1) A mixture design method which is based on the indirect tensile test should be developed in the pavement design procedures.

(2) Additional research should be conducted to evaluate the effect of soil binder content for additional types of aggregate.

(3) The adverse effects of moisture and its relationship with soil binder content should be investigated in more detail.

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APPENDIX A

MIXTURE GRADATIONS

TABLE A1. EAGLE LAKE MATERIAL GRADATION-
0 PERCENT SOIL BINDER

Sieve Size	Individual Cumulative % Total Mixture Retained				Total Cumulative % Retained
	Lone Star Coarse Sand	Lone Star Gem Sand	Tanner Walker Sand	Stiles Coarse Sand	
1 3/4"	0.0				0.0
1 1/4"	4.9				4.9
1"	21.6		0.0	0.0	21.6
7/8"	26.9		0.6	0.1	27.6
5/8"	38.3	0.0	0.7	0.2	39.2
1/2"	45.1	0.2	1.0	0.3	46.6
3/8"	51.4	0.3	1.2	0.4	53.3
#4	59.2	11.8	1.8	1.1	73.9
#10	61.3	17.1	2.9	3.3	84.6
#20	61.5	17.2	4.8	7.0	90.5
#40	61.6	17.4	11.0	10.0	100.0
#80	0.0	0.0	0.0	0.0	100.0
#200	0.0	0.0	0.0	0.0	100.0
% Passing #200	0.0	0.0	0.0	0.0	0.0

TABLE A2. EAGLE LAKE MATERIAL GRADATION-
5 PERCENT SOIL BINDER

Sieve Size	Lone Star Coarse Sand	Lone Star Gem Sand	Tanner Walker Sand	Stiles Coarse Sand	Total Cumulative % Retained
	Individual Cumulative % Total Mixture Retained				
1 3/4"	0.0				0.0
1 1/4"	4.6				4.6
1"	20.5		0.0	0.0	20.5
7/8"	25.6		0.6	0.1	26.3
5/8"	36.4	0.0	0.7	0.2	37.3
1/2"	42.8	0.1	1.0	0.3	44.2
3/8"	48.9	0.2	1.1	0.4	50.6
#4	56.3	11.1	1.7	1.1	70.2
#10	58.3	16.2	2.8	3.1	80.4
#20	58.4	16.3	4.6	6.7	86.0
#40	58.5	16.5	10.4	9.6	95.0
#80	58.5	16.5	13.5	10.0	98.5
#200	58.5	16.5	14.8	10.1	99.9
% Passing #200	0.0	0.0	0.1	0.0	0.1

TABLE A3. EAGLE LAKE MATERIAL GRADATION-
10 PERCENT SOIL BINDER

Sieve Size	Individual Cumulative % Total Mixture Retained				Total Cumulative % Retained
	Lone Star Coarse Sand	Lone Star Gem Sand	Tanner Walker Sand	Stiles Coarse Sand	
1 3/4"	0.0				0.0
1 1/4"	4.4				4.4
1"	19.4		0.0	0.0	19.4
7/8"	24.2		0.5	0.1	24.8
5/8"	34.4	0.0	0.6	0.2	35.2
1/2"	40.5	0.1	0.9	0.3	41.8
3/8"	46.2	0.2	1.0	0.4	47.8
#4	53.2	10.6	1.5	1.1	66.4
#10	55.2	15.4	2.6	3.0	76.2
#20	55.3	15.5	4.3	6.4	81.5
#40	55.4	15.6	9.9	9.1	90.0
#80	55.4	15.6	16.1	10.0	97.1
#200	55.4	15.6	18.7	10.1	99.8
% Passing #200	0.1	0.0	0.1	0.0	0.2

TABLE A4. EAGLE LAKE MATERIAL GRADATION-
20 PERCENT SOIL BINDER

Sieve Size	Individual Cumulative % Total Mixture Retained				Total Cumulative % Retained
	Lone Star Coarse Sand	Lone Star Gem Sand	Tanner Walker Sand	Stiles Coarse Sand	
1 3/4"	0.0				0.0
1/4"	3.9				3.9
1"	17.2		0.0	0.0	17.2
7/8"	21.5		0.5	0.1	22.1
5/8"	30.6	0.0	0.6	0.2	31.4
1/2"	36.0	0.1	0.8	0.3	37.2
3/8"	41.1	0.2	0.9	0.4	42.6
#4	47.3	9.4	1.4	1.0	59.1
#10	49.0	13.7	2.3	2.7	67.7
#20	49.1	13.8	3.8	5.7	72.4
#40	49.2	13.9	8.7	8.1	79.9
#80	49.2	13.9	21.2	9.8	94.1
#200	49.2	13.9	26.4	9.9	99.4
% Passing #200	0.1	0.1	0.3	0.1	0.6

TABLE A5. EAGLE LAKE MATERIAL GRADATION-
30 PERCENT SOIL BINDER

Sieve Size	Individual Cumulative % Total Mixture Retained				Total Cumulative % Retained
	Lone Star Coarse Sand	Lone Star Gem Sand	Tanner Walker Sand	Stiles Coarse Sand	
1 3/4"	0.0				0.0
1 1/4"	3.4				3.4
1"	15.0		0.0	0.0	15.0
7/8"	18.7		0.4	0.1	19.2
5/8"	26.6	0.0	0.5	0.2	27.3
1/2"	31.3	0.1	0.7	0.3	32.4
3/8"	35.7	0.2	0.8	0.4	37.1
#4	41.1	8.2	1.2	0.9	51.4
#10	42.6	11.9	2.0	2.4	58.9
#20	42.7	12.0	3.3	5.0	63.0
#40	42.8	12.1	7.6	7.1	69.6
#80	42.8	12.1	26.5	9.7	91.1
#200	42.8	12.1	34.4	9.9	99.2
% Passing #200	0.2	0.1	0.4	0.1	0.8

APPENDIX B

PREPARATION OF SAMPLE FOR MOISTURE DAMAGE

APPENDIX B. PREPARATION OF SAMPLE FOR MOISTURE DAMAGE

I. First Day

1. Specimens were compacted according to Tex 126-E.
2. The heights of specimens in the mold were measured.
3. The specimens were allowed to cool in the mold for 30 to 60 minutes before ejecting from the mold.
4. The specimens including material scraped from the mold were weighed.
5. The dimensions of the specimen were measured.
6. The specimens were cured overnight at room temperature.

II. Second Day

1. The compacted specimens were sawed to produce two indirect tensile test specimens.
2. The densities were obtained by weighing and measuring the specimens.
3. The sawed faces of the specimens were coated with a thin coat of hot asphalt.
4. The specimens were weighed and stored overnight at room temperature i.e. approximately 24°C (75°F).

III. Third Day

1. The specimens were pressure wetted for 15 minutes at a hydrostatic water pressure of 8265 kPa (1200 psi) at a water temperature of 65°C (150°F).
2. The specimens were weighed after surface drying.
3. The specimens were placed in a water bath at 24°C (75°F).
4. The specimens were removed from the waterbath, surface dried, and weighed.
5. Specimens were tested at 24°C (75°F).

APPENDIX C

SUMMARY OF STATIC TEST CHARACTERISTICS

TABLE C1. STATIC TEST CHARACTERISTICS OF EAGLE LAKE GRAVEL MIXTURES

Soil Binder Content, %	Asphalt Content, %	Density, kg/m^3	Density, (pcf)	Air Void Content, %	Tensile Strength, kPa	Tensile Strength, (psi)	Static Modulus of Elasticity E_S, kPa	Static Modulus of Elasticity E_S, (psi)	Poisson's Ratio
0	2.5	2,313	(144.4)	8.29	508	(74.0)	438,000	(64,000)	0.30
		2,350	(146.7)	6.88	693	(100.0)	1,001,000	(145,000)	0.54
	2.75	2,360	(147.3)	6.13	733	(106.0)	1,128,000	(164,000)	0.37
		2,353	(146.9)	6.38	863	(125.0)	1,401,000	(203,000)	0.45
	3.0	2,307	(144.0)	7.89	707	(103.0)	835,000	(121,000)	0.48
		2,347	(146.5)	6.28	1,020	(148.0)	1,202,000	(174,000)	0.33
		2,356	(147.1)	5.87	712	(103.0)	1,227,000	(178,000)	0.32
		2,393	(149.4)	4.42	1,158	(168.0)	1,837,000	(266,000)	0.47
	3.5	2,371	(148.0)	4.60	963	(140.0)	1,251,000	(182,000)	0.32
		2,390	(149.2)	3.82	1,107	(161.0)	1,511,000	(219,000)	0.51
		2,368	(147.8)	4.71	816	(118.0)	754,000	(109,000)	0.51
		2,387	(149.0)	3.98	1,085	(157.0)	1,473,000	(214,000)	0.27
	4.0	2,388	(149.1)	3.20	936	(136.0)	1,355,000	(196,000)	0.18
		2,385	(148.9)	3.32	1,021	(148.0)	660,000	(96,000)	0.47
5	2.5	2,384	(148.3)	5.62	985	(143.0)	1,913,000	(277,000)	0.52
		2,361	(147.4)	6.49	947	(137.0)	1,352,000	(196,000)	0.37
	2.75	2,398	(149.7)	4.66	1,217	(176.0)	2,022,000	(293,000)	0.46
		2,390	(149.2)	5.00	1,197	(174.0)	1,358,000	(197,000)	0.28
	3.0	2,427	(151.5)	3.17	1,337	(194.0)	2,162,000	(314,000)	0.30
		2,411	(150.5)	3.77	1,400	(203.0)	2,102,000	(305,000)	0.41
	3.5	2,408	(150.3)	3.21	1,118	(162.0)	1,478,000	(214,000)	0.44
		2,403	(150.0)	3.38	1,152	(167.0)	1,366,000	(198,000)	0.29

(continued)

TABLE C1. Continued

Soil Binder Content, %	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	2.75	2,411	(150.5)	4.19	1,118	(162.0)	840,000	(122,000)	0.57
		2,390	(149.2)	5.04	1,110	(161.0)	816,000	(118,000)	0.63
	3.0	2,412	(150.6)	3.78	1,318	(191.0)	1,376,000	(200,000)	0.36
		2,392	(149.3)	4.61	1,104	(160.0)	1,678,000	(243,000)	0.37
	3.5	2,416	(150.8)	2.93	1,095	(159.0)	1,251,000	(181,000)	0.26
		2,385	(148.9)	4.17	913	(132.0)	601,000	(87,000)	0.17
	4.0	2,361	(147.4)	4.42	728	(106.0)	340,000	(49,000)	0.24
		2,360	(147.3)	4.49	650	(94.0)	259,000	(38,000)	0.23
20	3.0	2,372	(148.1)	5.52	891	(129.0)	688,000	(100,000)	0.99
		2,340	(146.1)	6.83	866	(126.0)	1,000,000	(145,000)	0.62
	3.5	2,408	(150.3)	3.43	1,342	(195.0)	1,110,000	(161,000)	0.91
		2,379	(148.5)	4.55	1,185	(172.0)	903,000	(131,000)	0.65
		2,400	(149.8)	3.75	1,096	(159.0)	845,000	(122,000)	0.73
		2,388	(149.1)	4.20	1,004	(145.0)	753,000	(109,000)	0.57
	4.0	2,387	(149.0)	3.52	1,035	(150.0)	1,051,000	(152,000)	0.13
		2,427	(151.5)	1.90	1,061	(154.0)	954,000	(138,000)	0.25
	4.5	2,352	(146.8)	4.22	884	(128.0)	886,000	(128,000)	0.04
		2,351	(146.8)	4.27	834	(121.0)	894,000	(130,000)	0.22
30	3.5	2,332	(145.6)	6.30	1,080	(156.0)	1,900,000	(276,000)	0.50
		2,324	(145.1)	6.50	880	(128.0)	1,810,000	(262,000)	0.45
	4.0	2,363	(147.5)	4.30	1,250	(181.0)	2,606,000	(378,000)	0.39
		2,363	(147.5)	4.30	1,120	(163.0)	2,081,000	(302,000)	0.61
	4.5	2,366	(147.7)	3.4	990	(144.0)	1,112,000	(161,000)	0.35
		2,382	(148.7)	2.8	880	(128.0)	996,000	(144,000)	0.37
	5.0	2,335	(145.8)	4.0	740	(108.0)	635,000	(92,000)	-0.02
		2,348	(146.6)	3.5	740	(108.0)	512,000	(74,000)	0.26

(continued)

TABLE C2. STATIC TEST CHARACTERISTICS OF LUBBOCK LIMESTONE MIXTURES

Soil Binder Content, %	Asphalt Content, %	Density, kg/m^3	Density, (pcf)	Air Void Content, %	Tensile Strength, kPa	Tensile Strength, (psi)	Static Modulus of Elasticity E_S, kPa	Static Modulus of Elasticity E_S, (psi)	Poisson's Ratio
0	5.5	2,121	(132.4)	14.14	935	(136.0)	1,005,000	(146,000)	0.15
		2,151	(134.3)	12.88	1,027	(149.0)	856,000	(124,000)	0.10
	6.0	2,177	(135.9)	11.21	1,154	(167.0)	1,400,000	(203,000)	0.17
		2,188	(136.6)	10.71	1,060	(154.0)	1,054,000	(153,000)	0.10
	6.5	2,138	(133.5)	12.13	915	(133.0)	1,038,000	(151,000)	0.40
		2,164	(135.1)	11.08	1,025	(149.0)	1,515,000	(220,000)	0.18
	7.0	2,188	(136.6)	9.39	946	(137.0)	937,000	(135,000)	0.18
		2,223	(138.8)	7.93	1,005	(146.0)	1,024,000	(148,000)	0.17
	7.5	2,164	(135.1)	9.76	925	(134.0)	719,000	(104,000)	0.11
		2,211	(138.0)	7.85	944	(137.0)	714,000	(104,000)	0.15
5	5.0	2,094	(130.7)	15.80	969	(141.0)	1,006,000	(146,000)	0.07
		2,145	(133.9)	13.80	1,029	(149.0)	1,256,000	(182,000)	0.03
	5.5	2,135	(133.3)	13.50	1,095	(159.0)	1,305,000	(189,000)	0.31
		2,137	(133.4)	13.50	1,214	(176.0)	1,350,000	(196,000)	0.12
	6.0	2,175	(135.8)	11.20	1,393	(202.0)	1,397,000	(203,000)	0.22
		2,169	(135.4)	11.50	1,364	(198.0)	1,653,000	(240,000)	0.23
	6.5	2,195	(137.0)	9.80	1,137	(165.0)	1,383,000	(201,000)	0.19
		2,211	(138.0)	9.10	1,074	(156.0)	1,184,000	(172,000)	0.18

(continued)

TABLE C2. Continued

Soil Binder Content, %	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	5.0	2,089	(130.4)	16.01	881	(127.8)	970,077	(140,700)	0.21
		2,164	(135.1)	13.01	1,214	(176.0)	1,379,619	(200,100)	0.16
	5.5	2,179	(136.0)	11.81	1,331	(193.1)	1,545,091	(224,100)	0.19
		2,163	(135.0)	12.44	1,233	(178.8)	1,435,466	(208,200)	0.09
		2,172	(135.6)	12.03	1,377	(199.7)	1,465,113	(212,500)	0.17
		2,212	(138.1)	10.42	1,451	(210.4)	1,609,211	(233,400)	0.26
	6.0	2,132	(133.1)	13.05	1,202	(174.4)	1,616,795	(234,500)	0.36
		2,203	(137.5)	10.15	1,469	(213.1)	2,203,530	(319,600)	0.35
	6.5	2,206	(137.7)	9.35	1,360	(197.3)	1,622,311	(235,300)	0.51
		2,228	(139.1)	8.41	1,220	(176.9)	1,965,665	(285,100)	0.48
	7.0	2,206	(137.7)	8.65	1,027	(149.0)	961,804	(139,500)	0.23
		2,217	(138.4)	8.22	987	(143.2)	994,898	(144,300)	0.36
25	6.0	2,134	(133.2)	12.7	1,230	(179.0)	1,170,000	(170,000)	0.17
		2,147	(134.0)	12.1	1,190	(173.0)	998,000	(145,000)	0.24
	6.5	2,138	(135.5)	11.8	1,190	(173.0)	1,339,000	(197,000)	0.16
		2,142	(133.7)	11.7	1,120	(163.0)	942,000	(137,000)	0.41
		2,135	(133.3)	12.0	1,080	(157.0)	972,000	(141,000)	0.44
		2,158	(134.7)	11.1	1,140	(165.0)	981,000	(142,000)	0.36
	7.0	2,167	(135.3)	10.0	1,270	(184.0)	1,652,000	(240,000)	0.70
		2,161	(134.9)	10.2	1,250	(181.0)	831,000	(121,000)	0.21
		2,172	(135.6)	9.8	1,190	(172.0)	915,000	(133,000)	0.39
	7.5	2,182	(136.2)	8.7	900	(131.0)	746,000	(108,000)	0.34
		2,204	(137.6)	7.8	860	(125.0)	712,000	(103,000)	0.30
		2,180	(136.1)	8.8	970	(141.0)	1,278,000	(185,000)	0.37
		2,179	(136.0)	9.0	970	(141.0)	1,007,000	(146,000)	0.25
	8.0	2,190	(136.7)	7.8	700	(102.0)	952,000	(138,000)	0.18
		2,188	(136.6)	7.8	740	(107.0)	750,000	(109,000)	0.41
	8.5	2,179	(136.0)	7.5	570	(82.0)	400,000	(58,000)	0.19
		2,190	(136.7)	7.1	530	(77.0)	849,000	(123,000)	0.31

APPENDIX D

SUMMARY OF REPEATED LOAD CHARACTERISTICS

TABLE D1. REPEATED LOAD CHARACTERISTICS OF EAGLE LAKE GRAVEL MIXTURES

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity E_R, kPa	(psi)	Poisson's Ratio
0	70	(10.2)	3.0	2,351	(146.8)	6.1	7,263	2,127,000	(309,000)	.09
				2,342	(146.2)	6.5	1,968	2,040,000	(296,000)	.19
				2,362	(147.4)	5.7	5,080	2,485,000	(360,000)	.72
			3.5	2,397	(149.6)	3.5	17,927	2,012,000	(292,000)	.00
				2,385	(148.9)	4.0	33,898	2,130,000	(309,000)	-.11
				2,379	(148.5)	4.3	29,140	1,963,000	(285,000)	.02
			4.0	2,384	(148.8)	3.4	17,533	2,384,000	(340,000)	.27
				2,383	(148.8)	3.4	30,016	1,824,000	(265,000)	.17
			4.5	2,344	(146.3)	4.3	10,477	1,801,000	(261,000)	.23
				2,359	(147.3)	3.7	13,256	1,965,000	(285,000)	.24
	120	(17.4)	3.0	2,340	(146.0)	6.6	784	1,829,000	(265,000)	.47
				2,364	(147.6)	5.6	1,331	1,943,000	(283,000	.07
				2,360	(147.3)	5.8	2,759	2,259,000	(328,000)	.11
			3.5	2,403	(150.0)	3.3	5,826	2,725,000	(395,000)	.07
				2,382	(148.0)	4.2	3,515	1,982,000	(287,000)	.11
				2,370	(148.0)	4.6	1,280	2,137,000	(310,000)	.24
			4.0	2,370	(147.9)	3.9	3,018	2,340,000	(339,000)	.22
				2,375	(148.3)	3.7	3,678	2,362,000	(343,000)	.40
			4.5	2,328	(145.3)	4.9	1,267	1,639,000	(238,000)	.29
				2,346	(146.4)	4.2	1,977	2,095,000	(304,000)	.27

(continued)

TABLE D1. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity E_R, kPa	(psi)	Poisson's Ratio
5	80	(11.6)	2.75	2,407	(150.3)	4.3	28,595	2,805,000	(407,000)	-.10
			3.0	2,421	(151.1)	3.4	46,138	4,029,000	(584,000)	.08
				2,392	(149.3)	4.6	11,417	5,055,000	(733,000)	.44
			3.5	2,396	(149.6)	3.7	48,647	6,119,000	(888,000)	.22
				2,391	(149.3)	3.9	39,915	5,243,000	(760,000)	.35
			4.0	2,376	(148.4)	3.7	9,497	3,767,000	(546,000)	.41
				2,368	(147.8)	4.1	10,565	4,451,000	(646,000)	.50
	120	(17.4)	2.75	2,395	(149.5)	4.8	2,724	3,210,000	(466,000)	.09
			3.0	2,403	(150.0)	4.1	3,790	6,132,000	(889,000)	.56
				2,376	(148.3)	5.2	2,063	4,478,000	(650,000)	.49
			3.5	2,406	(150.2)	3.3	7,353	4,320,000	(627,000)	.22
				2,406	(150.2)	3.3	12,401	2,813,000	(408,000)	.05
			4.0	2,390	(149.2)	3.2	4,470	3,613,000	(524,000)	.49
				2,369	(147.9)	4.0	2,344	3,205,000	(465,000)	.55

(continued)

TABLE D1. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity E_R, kPa	(psi)	Poisson's Ratio
10	40	(5.8)	2.75	2,399	(149.8)	4.7	21,010	2,714,000	(394,000)	.11
				2,381	(148.6)	5.4	29,476	2,165,000	(314,000)	-.03
				2,358	(147.2)	6.3	11,422	1,729,000	(251,000)	.10
			3.0	2,384	(148.8)	4.9	62,868	2,169,000	(315,000)	-.02
				2,396	(149.6)	4.4	16,406	2,072,000	(301,000)	.02
			3.5	2,397	(149.6)	3.7	126,748	1,137,000	(165,000)	-.04
				2,397	(149.6)	3.7	80,966	1,980,000	(287,000)	.02
			4.0	2,370	(147.9)	4.1	39,775	1,127,000	(164,000)	.19
				2,383	(148.8)	3.5	83,820	1,457,000	(211,000)	-.16
	120	(17.4)	2.75	2,386	(149.0)	5.2	5,955	2,252,000	(327,000)	-.08
				2,374	(148.2)	5.7	884	1,763,000	(256,000)	.09
				2,382	(148.7)	5.4	1,050	2,192,000	(318,000)	.08
			3.0	2,375	(148.3)	5.3	2,209	2,046,000	(297,000)	-.00
				2,399	(149.8)	4.3	5,717	1,914,000	(278,000)	-.00
			3.5	2,392	(149.4)	3.9	4,594	2,309,000	(335,000)	.04
				2,394	(149.5)	3.8	3,377	2,250,000	(326,000)	.07
			4.0	2,377	(148.4)	3.8	2,366	2,001,000	(290,000)	.06
				2,380	(148.6)	3.7	2,082	2,311,000	(355,000)	.02

(continued)

TABLE D1. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity E_R, kPa	(psi)	Poisson's Ratio
20	40	(5.8)	3.0	2,364	(147.6)	5.9	8,936	2,306,000	(334,000)	.06
				2,367	(147.8)	5.7	5,994	2,392,000	(347,000)	.16
			3.5	2,360	(147.3)	5.3	17,740	2,009,000	(291,000)	.13
				2,399	(149.8)	3.8	72,433	2,133,000	(309,000)	.04
				2,371	(148.0)	4.9	27,236	1,982,000	(287,000)	-.02
			4.0	2,386	(149.0)	3.6	75,000	2,394,000	(347,000)	.00
				2,368	(147.9)	4.3	48,641	1,510,000	(219,000)	-.04
			4.5	2,357	(147.1)	4.0	44,167	1,513,000	(219,000)	.00
				2,372	(148.1)	3.4	40,346	2,238,000	(325,000)	.06
	120	(17.4)	3.0	2,332	(145.6)	7.1	735	2,140,000	(310,000)	.14
				2,340	(146.1)	6.8	812	2,091,000	(303,000)	.09
			3.5	2,394	(149.5)	4.0	2,925	1,762,000	(256,000)	.11
				2,394	(149.4)	4.0	3,052	2,139,000	(310,000)	.08
				2,396	(149.6)	3.9	3,296	2,500,000	(363,000)	-.02
			4.0	2,365	(147.6)	4.4	3,510	2,004,000	(291,000)	.10
				2,364	(147.6)	4.5	3,085	1,958,000	(284,000)	.05
			4.5	2,371	(148.0)	3.5	1,279	2,037,000	(296,000)	.37
				2,353	(146.9)	4.2	1,349	2,001,000	(290,000)	.19
				2,350	(146.7)	4.3	1,300	2,311,000	(335,000)	.16

(continued)

TABLE D1. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity* E_R, kPa	(psi)	Poisson's Ratio
30	40	(5.8)	3.5	2,343	(146.3)	5.8	13,072	2,550,000	(370,000)	.52
				2,352	(146.8)	5.4	11,111	3,129,000	(454,000)	.08
				2,329	(145.4)	6.4	7,636	3,202,000	(464,000)	.34
			4.0	2,374	(148.2)	3.8	47,869	3,272,000	(474,000)	-.08
				2,347	(146.5)	4.9	12,500	4,703,000	(682,000)	1.25
				2,352	(146.8)	4.7	12,540	3,148,000	(456,000)	.11
			4.5	2,360	(147.3)	3.7	40,240	2,077,000	(301,000)	.01
				2,355	(147.0)	3.9	40,671	2,527,000	(366,000)	.00
			5.0	2,342	(146.2)	3.8	30,700	4,376,000	(635,000)	.91
				2,348	(146.6)	3.5	15,920	8,699,000	(1,261,000)	2.13
	120	(17.4)	3.5	2,343	(146.3)	5.8	1,121	3,124,000	(453,000)	.74
				2,331	(145.5)	6.3	869	3,355,000	(486,000)	.84
			4.0	2,345	(146.4)	5.0	1,355	3,418,000	(496,000)	.69
				2,366	(147.7)	4.1	2,109	2,443,000	(354,000)	.07
			4.5	2,362	(147.5)	3.6	2,308	3,439,000	(499,000)	.80
				2,356	(147.1)	3.8	1,808	2,825,000	(410,000)	.42
			5.0	2,334	(145.7)	4.1	2,044	3,350,000	(486,000)	.97
				2,363	(147.5)	2.9	1,695	4,124,000	(598,000)	1.57

*For cycle corresponding to approximately 50 percent of the fatigue life.

(continued)

TABLE D2. REPEATED LOAD CHARACTERISTICS OF LUBBOCK LIMESTONE MIXTURES

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity* E_R, kPa	(psi)	Poisson's Ratio
0	150	(21.8)	6.0	2,135	(133.3)	12.9	18,138	1,988,000	(288,000)	.05
				2,195	(137.0)	10.5	15,243	3,847,000	(558,000)	.20
			6.5	2,200	(137.3)	9.6	31,944	2,550,000	(370,000)	.09
				2,198	(137.2)	9.7	22,735	2,628,000	(381,000)	.25
				2,145	(133.9)	11.8	23,455	2,893,000	(410,000)	.16
			7.0	2,171	(135.5)	10.2	8,083	2,056,000	(298,000)	.22
				2,176	(135.8)	9.9	10,762	2,719,000	(394,000)	.19
	250	(36.3)	6.0	2,169	(135.4)	11.5	3,721	2,650,000	(384,000)	.23
				2,166	(135.2)	11.7	3,760	1,856,000	(269,000)	.01
			6.5	2,163	(135.0)	11.1	4,028	1,991,000	(289,000)	.00
				2,171	(135.5)	10.8	2,007	2,244,000	(326,000)	.17
				2,172	(135.6)	10.7	5,513	2,981,000	(432,000)	.21
			7.0	2,203	(137.5)	8.8	1,330	2,325,000	(337,000)	.27
				2,121	(132.4)	12.2	1,450	2,291,000	(332,000)	.22
5	200	(29.0)	5.5	2,083	(130.0)	15.7	6,072	1,791,000	(260,000)	.00
				2,182	(136.2)	11.6	30,035	3,167,000	(459,000)	.12
			6.0	2,201	(137.4)	10.2	44,183	2,718,000	(394,000)	.03
				2,180	(136.1)	11.1	32,016	3,335,000	(484,000)	.11
			6.5	2,188	(136.6)	10.1	7,372	2,921,000	(325,000)	.19
				2,161	(134.9)	11.2	31,628	2,880,000	(418,000)	.11
	300	(43.5)	5.5	2,105	(131.4)	14.8	2,195	2,669,000	(387,000)	.22
				2,156	(134.6)	12.7	3,698	3,052,000	(443,000)	.22
			6.0	2,192	(136.8)	10.6	4,329	3,143,000	(456,000)	.27
				2,212	(138.1)	9.7	5,632	4,277,000	(620,000)	.34
				2,172	(135.6)	11.4	4,317	3,297,000	(478,000)	.23
			6.5	2,190	(136.7)	10.0	1,328	3,182,000	(461,000)	.39
				2,187	(136.5)	10.2	4,692	3,681,000	(534,000)	.13

(continued)

TABLE D2. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m³	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity* E_R, kPa	(psi)	Poisson's Ratio
			4.0	2,106	(131.5)	16.6	8,851	1,810,000	(262,493)	.07
				2,114	(132.0)	16.3	15,087	2,012,000	(291,794)	.17
			4.5	2,182	(136.2)	13.0	48,018	2,441,000	(354,012)	.04
				2,147	(134.0)	14.4	49,744	2,640,00	(382,919)	.05
			5.0	2,164	(135.1)	13.0	62,500	3,065,000	(444,595)	.13
				2,196	(137.1)	11.7	50,725	3,158,000	(458,011)	.05
				2,151	(134.3)	13.6	30,402	4,343,000	(629,952)	.30
				2,208	(137.8)	11.3	43,738	3,922,000	(568,916)	.19
	170	(24.7)	5.5	2,185	(136.4)	11.5	20,350	3,089,000	(448,002)	.30
				2,185	(136.4)	11.5	27,103	3,914,000	(567,620)	.41
				2,168	(135.3)	12.3	36,168	2,679,000	(388,605)	.23
			6.0	2,196	(137.1)	10.4	22,413	4,017,000	(582,645)	.24
				2,190	(136.7)	10.7	24,109	2,679,000	(388,549)	.20
				2,159	(134.8)	11.9	35,387	2,827,000	(410,087)	.01
			6.5	2,207	(137.8)	9.3	23,653	3,212,000	(466,000)	.29
				2,212	(138.1)	9.1	19,150	4,450,000	(645,000)	.52
10										
			4.0	2,116	(132.1)	16.2	3,291	1,818,000	(264,000)	.13
				2,122	(132.5)	16.0	1,816	1,782,000	(258,000)	.03
			4.5	2,183	(136.3)	12.9	5,453	2,833,000	(411,000)	.25
				2,145	(133.9)	14.4	3,090	2,506,000	(363,000)	.11
			5.0	2,176	(135.8)	12.6	4,990	3,680,000	(534,000)	.28
				2,192	(136.8)	11.9	6,405	3,629,000	(526,000)	.29
				2,145	(133.9)	13.8	4,979	3,680,000	(534,000)	.28
	270	(39.2)	5.5	2,187	(136.5)	11.5	4,062	3,027,000	(439,000)	.41
				2,214	(138.2)	10.4	6,068	4,508,000	(654,000)	.52
				2,196	(137.1)	11.1	7,513	5,595,000	(812,000)	.38
			6.0	2,188	(136.6)	10.8	4,635	2,947,000	(427,000)	.19
				2,177	(135.9)	11.2	2,124	3,028,000	(439,000)	.36
				2,209	(137.9)	9.9	7,816	4,771,000	(692,000)	.28
			6.5	2,177	(135.9)	10.5	2,670	2,943,000	(427,000)	.31
				2,216	(138.3)	9.0	3,181	3,620,000	(525,000)	.49
				2,233	(139.4)	8.2	1,937	4,013,000	(582,000)	.57

(continued)

TABLE D2. Continued

Soil Binder Content, %	Stress Level, kPa	(psi)	Asphalt Content, %	Density, kg/m^3	(pcf)	Air Void Content, %	Fatigue Life N_f, Cycles	Resilient Modulus of Elasticity* E_R, kPa	(psi)	Poisson's Ratio
25	150	(21.7)	6.5	2,153	(134.4)	11.2	21,088	2,352,000	(341,000)	0.07
			7.0	2,148	(134.1)	10.8	18,570	2,221,000	(322,000)	0.13
				2,161	(134.9)	10.2	27,150	2,625,000	(381,000)	0.07
				2,207	(137.8)	8.3	39,381	3,759,000	(545,000)	0.16
			7.5	2,182	(136.2)	8.7	27,500	2,726,000	(395,000)	0.12
				2,196	(137.1)	8.1	40,000	2,306,000	(334,000)	0.01
				2,188	(136.6)	8.4	73,144	2,674,000	(388,000)	0.10
			8.0	2,180	(136.1)	8.1	24,663	2,898,000	(420,000)	0.03
				2,177	(135.9)	8.2	12,630	2,562,000	(372,000)	0.25
	250	(36.2)	6.5	2,135	(133.3)	11.9	2,768	2,095,000	(304,000)	0.09
			7.0	2,175	(135.8)	9.6	3,312	2,843,000	(412,000)	0.13
				2,169	(135.4)	9.9	4,382	2,974,000	(431,000)	0.21
				2,175	(135.8)	9.6	4,880	2,985,000	(433,000)	-0.08
			7.5	2,178	(136.0)	8.8	3,070	3,339,000	(484,000)	0.33
				2,187	(136.5)	8.5	3,118	2,802,000	(406,000)	0.14
				2,198	(137.2)	8.0	4,881	2,352,000	(341,000)	0.13
			8.0	2,178	(136.0)	8.2	3,448	3,019,000	(438,000)	0.33
				2,185	(136.4)	7.9	2,066	2,610,000	(379,000)	0.09

APPENDIX E

SUMMARY OF PERMANENT DEFORMATION CHARACTERISTICS

TABLE E1. PERMANENT DEFORMATION CHARACTERISTICS OF EAGLE LAKE MIXTURES.

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
				7,263	266.2	(10.5)	558.8	(22.0)
			3.0	1,968	311.9	(12.3)	2,350.0	(92.5)
				5,080	267.2	(10.5)	852.4	(33.6)
				17,927	379.0	(14.9)	263.1	(10.4)
			3.5	33,898	322.1	(12.7)	103.5	(4.1)
	70	(10.2)		29,140	376.9	(14.8)	145.6	(5.7)
			4.0	17,533	493.8	(19.4)	317.0	(12.5)
				30,016	464.3	(18.3)	146.1	(5.8)
			4.5	10,477	982.5	(38.7)	1,025.0	(40.4)
				13,256	795.5	(31.3)	630.2	(24.8)
0								
				784	287.5	(11.3)	5,723.0	(225.3)
			3.0	1,331	318.0	(12.5)	3,602.0	(141.8)
				2,759	308.9	(12.2)	3,048.0	(120.0)
				5,826	291.6	(11.5)	826.0	(32.5)
	120	(17.4)	3.5	3,515	438.9	(17.3)	1,626.0	(64.0)
				1,280	337.3	(13.3)	4,018.0	(158.2)
			4.0	3,018	463.3	(18.3)	1,993.0	(78.5)
				3,678	495.0	(19.5)	1,524.0	(60.0)
			4.5	1,267	1,051.6	(41.4)	1,261.0	(496.3)
				1,977	731.5	(28.8)	5,170.0	(203.5)

(continued)

TABLE E1. Continued

Soil Binder Content %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
5	80	(11.6)	2.75	28,595	260.1	(10.3)	117.1	(4.6)
			3.0	46,138	249.9	(9.8)	75.6	(3.0)
				11,417	282.4	(11.1)	368.0	(14.5)
			3.5	48,647	460.2	(18.1)	100.0	(3.9)
				39,915	529.3	(20.8)	133.4	(5.3)
			4.0	9,497	829.1	(32.6)	924.1	(36.4)
				10,565	657.4	(25.9)	812.8	(32.0)
	120	(17.4)	2.75	2,724	283.5	(11.1)	1,567.0	(61.7)
			3.0	3,790	317.0	(12.5)	1,094.0	(43.1)
				2,063	227.6	(9.0)	1,778.0	(70.0)
			3.5	7,353	497.8	(19.6)	845.6	(33.3)
				12,401	491.7	(19.4)	457.2	(18.0)
			4.0	4,470	602.5	(23.7)	1,821.0	(71.7)
				2,344	596.4	(23.5)	3,556.0	(140.0)

(continued)

TABLE E1. Continued

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
10	40	(5.8)	2.75	21,010	166.6	(6.6)	99.4	(3.9)
				29,476	173.7	(6.8)	80.7	(3.2)
				11,422	189.0	(7.4)	237.7	(9.4)
			3.0	62,868	214.4	(8.4)	43.3	(1.7)
				16,406	184.9	(7.3)	153.8	(6.1)
			3.5	126,748	363.7	(14.3)	29.5	(1.2)
				80,966	515.1	(20.3)	83.1	(3.3)
			4.0	39,775	816.9	(32.2)	238.9	(9.4)
				83,820	707.1	(27.8)	81.6	(3.2)
	120	(17.4)	2.75	5,955	277.4	(10.9)	668.8	(26.3)
				884	175.8	(6.9)	3,640.0	(143.3)
				1,050	202.2	(8.0)	3,226.0	(127.0)
			3.0	2,209	228.6	(9.0)	1,535.0	(60.4)
				5,717	359.7	(14.2)	923.5	(36.4)
			3.5	4,594	525.3	(20.7)	1,445.0	(56.9)
				3,377	443.0	(17.4)	1,872.0	(73.7)
			4.0	2,366	640.1	(25.2)	3,932.0	(154.8)
				2,082	706.1	(27.8)	5,032.0	(198.1)

(continued)

TABLE E1. Continued

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
20	40	(5.8)	3.0	5,994	180.8	(7.1)	486.2	(19.1)
				8,936	163.6	(6.4)	296.2	(11.7)
			3.5	17,740	242.8	(9.6)	204.6	(8.1)
				72,433	197.1	(7.8)	33.0	(1.3)
				27,236	204.2	(8.0)	103.5	(4.1)
			4.0	75,000	689.9	(27.2)	110.9	(4.4)
				48,641	493.8	(19.4)	120.0	(4.7)
			4.5	44,167	834.1	(32.9)	194.4	(7.7)
				40,346	815.8	(33.1)	237.1	(9.3)
	120	(17.4)	3.0	735	255.0	(10.0)	5,326.0	(209.7)
				812	233.7	(9.2)	4,910.0	(193.3)
			3.5	2,925	322.1	(12.6)	1,601.0	(63.0)
				3,058	375.9	(14.8)	1,878.0	(73.9)
				3,296	288.5	(11.4)	1,439.0	(56.6)
			4.0	3,510	823.0	(32.4)	3,426.0	(134.9)
				3,085	597.4	(23.5)	3,142.0	(123.7)
			4.5	1,279	831.1	(32.7)	9,689.0	(377.5)
				1,349	853.4	(33.6)	9,939.0	(391.3)
				1,300	631.9	(24.9)	7,971.0	(313.8)

(continued)

TABLE E1. Continued

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2} mm	(10^{-2} in.)	Rate of Vertical Permanent Deformation, 10^{-6} mm/cycle	10^{-6} (in./cycle)
				13,072	184.9	(7.3)	217.0	(8.5)
			3.5	11,111	185.9	(7.3)	234.4	(9.2)
				7,636	177.8	(7.0)	378.5	(14.9)
			4.0	12,500	189.0	(7.4)	250.9	(9.9)
	40	(5.8)		12.540	214.4	(8.4)	251.6	(9.9)
			4.5	40,671	440.9	(17.4)	148.1	(5.8)
				40,240	328.2	(12.9)	120.6	(4.7)
30			5.0	15,920	656.4	(26.0)	621.0	(24.5)
			3.5	1,121	239.8	(9.4)	3,586.0	(141.2)
				869	243.8	(9.6)	5,258.0	(207.0)
			4.0	1,355	291.6	(11.4)	3,658.0	(144.0)
	120	(17.4)		2,109	306.8	(12.1)	2,604.0	(102.5)
			4.5	2,308	473.5	(18.7)	3,487.0	(137.3)
				1,808	512.1	(20.2)	4,760.0	(187.4)
			5.0	1,695	558.8	(22.0)	5,753.0	(226.5)

TABLE E2. PERMANENT DEFORMATION CHARACTERISTICS OF LUBBOCK LIMESTONE MIXTURES

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2} mm	$(10^{-2}$ in.)	Rate of Vertical Permanent Deformation, 10^{-6} mm/cycle	10^{-6}(in./cycle)
0	150	(21.8)	6.0	18,138	134.1	(5.3)	88.1	(3.5)
				15,243	113.8	(4.5)	109.9	(4.3)
			6.5	31,944	238.8	(9.4)	90.4	(3.6)
				22,735	561.8	(22.1)	334.8	(13.2)
				23,455	214.4	(8.4)	103.6	(4.1)
			7.0	8,083	580.1	(22.8)	974.9	(38.4)
				10,762	508.0	(20.0)	688.8	(27.1)
	250	(36.3)	6.0	3,721	203.2	(8.0)	663.7	(26.1)
				3,760	194.1	(7.6)	596.1	(23.5)
			6.5	4,028	247.9	(9.8)	826.3	(32.5)
				2,007	550.7	(21.7)	3,772.0	(148.5)
				5,513	310.9	(12.2)	711.2	(28.0)
			7.0	1,330	643.1	(25.3)	6,429.0	(253.1)
				1,450	653.3	(25.7)	6,373.0	(250.9)
5	200	(29.0)	5.5	6,072	201.2	(7.9)	469.9	(18.5)
				30,035	179.8	(7.1)	89.3	(3.5)
			6.0	44,183	195.1	(7.7)	50.8	(2.0)
				32,016	375.9	(14.8)	129.3	(5.1)
			6.5	7,372	423.7	(16.7)	796.5	(31.4)
				31,628	165.6	(6.5)	59.4	(2.3)
	300	(43.5)	5.5	2,195	212.3	(8.4)	1,287.0	(50.6)
				3,698	167.6	(6.6)	589.3	(23.3)
			6.0	4,329	249.9	(9.8)	825.5	(32.5)
				5,632	161.5	(6.4)	420.4	(16.6)
				4,317	235.0	(9.2)	780.0	(30.7)
			6.5	1,328	578.1	(22.8)	6,281.0	(247.3)
				4,692	232.7	(9.2)	645.2	(25.4)

(continued)

TABLE E2. Continued

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
10	170	(24.7)	4.0	8,851	217.4	(8.6)	359.7	(14.2)
				15,087	119.9	(4.7)	80.6	(3.2)
			4.5	48,018	168.7	(6.6)	41.4	(1.6)
				49,744	112.8	(4.4)	31.7	(1.2)
			5.0	30,402	134.1	(5.3)	43.8	(1.7)
				43,738	178.8	(7.0)	54.2	(2.1)
			5.5	20,350	181.9	(7.2)	108.7	(4.3)
				27,103	141.2	(5.6)	65.6	(2.6)
				36,168	156.5	(6.2)	55.4	(2.2)
			6.0	22,413	165.6	(6.5)	87.1	(3.4)
				24,109	310.9	(12.2)	162.8	(6.4)
				35,387	121.9	(4.8)	37.7	(1.5)
			6.5	23,653	459.2	(18.1)	245.0	(9.6)
				19,150	312.9	(12.3)	228.7	(9.0)
	270	(39.2)	4.0	3,291	176.8	(7.0)	674.9	(26.6)
				1,816	153.4	(6.0)	838.2	(33.0)
			4.5	5,453	207.3	(8.2)	517.1	(20.4)
				3,090	191.0	(7.5)	773.7	(30.5)
			5.0	4,990	195.1	(7.7)	491.7	(19.4)
				6,405	120.9	(4.8)	237.1	(9.3)
				4,979	161.5	(6.4)	369.1	(14.5)
			5.5	4,062	185.9	(7.3)	631.4	(24.9)
				6,068	203.2	(8.0)	423.7	(16.7)
				7,513	148.3	(5.8)	260.3	(10.3)
			6.0	4,635	202.2	(8.0)	556.0	(21.9)
				2,124	239.8	(9.4)	1,580.0	(62.2)
				7,816	175.8	(6.9)	300.0	(11.8)
			6.5	2,670	359.7	(14.2)	1,783.0	(70.2)
				3,181	489.7	(19.3)	2,288.0	(90.1)
				1,937	277.4	(10.9)	1,969.0	(77.5)

(continued)

TABLE E2. Continued

Soil Binder Content, %	Applied Stress, kPa	(psi)	Asphalt Content, %	Fatigue Life N_f, Cycles	Vertical Permanent Deformation @ $0.5N_f$, 10^{-2}mm	(10^{-2}in.)	Rate of Vertical Permanent Deformation, 10^{-6}mm/cycle	10^{-6}(in./cycle)
25	150	(21.7)	6.5	21,088	110.7	(4.4)	49.7	(1.9)
			7.0	27,150	131.1	(5.1)	53.9	(2.1)
				18,570	115.8	(4.6)	69.9	(2.8)
				39,381	432.8	(17.0)	147.7	(5.8)
			7.5	27.500	581.2	(22.9)	287.8	(11.3)
				40,000	192.0	(7.6)	50.4	(1.9)
				73,144	191.0	(7.5)	33.2	(1.3)
			8.0	12,630	852.4	(33.6)	914.4	(36.0)
				24,663	539.5	(21.2)	322.6	(12.7)
	250	(36.2)	6.5	2,768	164.6	(6.5)	601.7	(23.7)
			7.0	3,312	151.4	(6.0)	593.9	(23.4)
				4,382	193.0	(7.6)	629.2	(24.8)
				4,880	402.3	(15.8)	1,225.0	(48.2)
			7.5	3,070	594.4	(23.4)	2,863.0	(112.7)
				4,881	344.4	(13.6)	877.6	(34.6)
				3,118	246.9	(9.7)	1,141.0	(44.9)
			8.0	2,066	692.9	(27.3)	5,182.0	(204.0)
				3,448	548.6	(21.6)	2,161.0	(85.1)