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16. Abstract

This report summarizes the results of an investigation of the fatigue and resilient characteristics of laboratory-prepared asphalt mixtures using the repeated-load indirect tensile test.

The mixtures studied contained crushed limestone or rounded gravel aggregates and an AC-10 asphalt cement ranging from 4 to 8 percent by weight of total mixture. The fatigue and repeated-load tests were conducted at 50, 75, and 100^{o} F, with the stress level ranging between 8 and 120 psi.

From the static tests, an equation which predicts indirect tensile strength in terms of testing temperature and mixture properties was developed. In addition, a strong relationship was found to exist between static modulus of elasticity and indirect tensile strength.

From the fatigue study, relationships between fatigue life and various test variables and mixture properties were evaluated and equations were developed for predicting fatigue life from initial strain and from the ratio of repeated tensile stress to the average indirect tensile strength.

Based on the repeated-load tests, the relationships between the number of repeated tensile stresses and various mixture properties, such as resilient and permanent strains, resilient modulus, modulus of cumulative total deformation, and resilient Poisson's ratio, were investigated and the possible use of these relationships for estimating the service life is discussed. The magnitudes and variations of average values of repeated-load resilient moduli and their rates of deterioration with repeated stress were estimated.

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FATIGUE AND RESILIENT CHARACTERISTICS OF ASPHALT MIXTURES BY REPEATED-LOAD INDIRECT TENSILE TEST

by

Adedare S. Adedimila
Thomas W. Kennedy

Research Report Number 183-5

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
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

August 1975

PREFACE

This is the fifth in a series of reports dealing with the findings of a research project concerned with tensile and elastic characterization of highway pavement materials. This report summarizes the results of a study in which the repeated-load indirect tensile test was utilized to determine the magnitude and variation of fatigue life and repeated-load elastic properties of laboratory-prepared asphalt mixtures.

Possible relationships between fatigue life and various mixture properties and testing variables, e.g., stress and testing temperature, were investigated and equations for fatigue life were developed. In addition, average values of the instantaneous and total resilient moduli and the deterioration in resilient modulus with load repetitions were evaluated.

This report was made possible through the combined efforts of a number of people. Special appreciation is extended to Messrs. James N. Anagnos, Pat Hardeman, Harold H. Dalrymple, and Victor N. Toth for their assistance in the testing program; to Messrs. Avery Smith, Gerald Peck, and James L. Brown, of the State Department of Highways and Public Transportation, who provided technical liason for the project; and to Messrs. Byron W. Porter and Calvin E. Cowher and Ms. Shirley Selz for their general assistance and consultation. Thanks are also due the Center for Highway Research staff who assisted in the preparation of the manuscript. The support of the Federal Highway Administration, Department of Transportation, is gratefully acknowledged.

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Thomas W. Kennedy

August 1975

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 a 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 Byron W. Porter and Thomas W. Kennedy, summarizes the results of a study comparing fatigue results of the dynamic 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.

ABSTRACT

This report summarizes the results of an investigation of the fatigue and resilient characteristics of laboratory-prepared asphalt mixtures using the repeated-load indirect tensile test.

The mixtures studied contained crushed limestone or rounded gravel aggregates and an AC-10 asphalt cement ranging from 4 to 8 percent by weight of total mixture. The fatigue and repeated-load tests were conducted at 50, 75, and 100° F, with the stress level ranging between 8 and 120 psi.

From the static tests, an equation which predicts indirect tensile strength in terms of testing temperature and mixture properties was developed. In addition, a strong relationship was found to exist between static modulus of elasticity and indirect tensile strength.

From the fatigue study, relationships between fatigue life and various test variables and mixture properties were evaluated and equations were developed for predicting fatigue life from initial strain and from the ratio of repeated tensile stress to the average indirect tensile strength.

Based on the repeated-load tests, the relationships between the number of repeated tensile stresses and various mixture properties, such as resilient and permanent strains, resilient modulus, modulus of cumulative total deformation, and resilient Poisson's ratio, were investigated and the probable use of these relationships for estimating the service life is discussed. The magnitudes and variations of average values of repeated-load resilient moduli and their rates of deterioration with repeated stress were estimated.

KEY WORDS: indirect tensile test, repeated-load, fatigue, resilient characteristics, asphalt mixtures.

SUMMARY

This report concerns the use of the repeated-load indirect tensile test for the study of the fatigue and resilient characteristics of laboratory-prepared asphalt mixtures. The variables considered include aggregate type, asphalt content, stress level, and testing temperature.

Static tests were conducted in order to obtain mixture properties for use in the evaluation and analysis of the fatigue and repeated-load test results. From a regression analysis, an equation which predicts indirect tensile strength in terms of testing temperature, asphalt content, and aggregate type was developed. In addition, a relationship between static modulus of elasticity and indirect tensile strength, which can be used to estimate the static modulus of elasticity, was developed.

From the repeated-load tests, the effects of stress, mixture variables, and testing temperature on fatigue life were evaluated and relationships between fatigue life and initial mixture strain and stress-strength ratio were developed.

The effects of repeated loads on the resilient and permanent strains were also evaluated and values of resilient modulus and Poisson's ratio were estimated. In addition, the rate of deterioration of resilient modulus with load applications was evaluated. Average values of modulus based on individual total and cumulative total deformations were also estimated and a comparison between the various moduli was made. Finally, the effects of repeated loads on Poisson's ratio were evaluated and a possible definition of the service life of blackbase specimens, based on the changes in the various properties with load applications, was suggested.

IMPLEMENTATION STATEMENT

The repeated-load indirect tensile test can be used to obtain the fatigue characteristics and resilient properties of asphalt mixtures. It is, therefore, recommended that the State Department of Highways and Public Transportation begin to use this test method.

Equations which relate fatigue life of asphalt mixtures to the initial mixture strain and stress-strength ratio can be used to obtain estimates of fatigue life. Where it is difficult or impractical to obtain deformation measurements, the relationship between tensile strength and static modulus of elasticity can be used to estimate the static modulus of elasticity if the tensile strength is known. The recommended equations can be solved directly with desk calculators or with the curves provided in the report.

Mixture design procedures should consider the fact that the optimum asphalt contents for maximum tensile strength, fatigue life, modulus of elasticity, and density are not necessarily the same. Thus, the mixture should be designed to produce the desired properties.

Additional study in the estimation and evaluation of the effects of repeated loads on the values of Poisson's ratio and the deterioration of resilient modulus is necessary.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

Numerous studies concerning the fatigue behavior of asphalt mixtures have been completed during the last two decades, and efforts have been made to incorporate the findings of these studies into pavement design procedures. The time-consuming aspect of fatigue testing has encouraged the development of fatigue life predictive equations in terms of applied repeated stresses and initial strains. However, much more research is needed in order to improve these equations.

New methods of pavement design based on elastic layered theory and slab theory have gained much recognition in recent years. The complicated analysis demanded in such theories has been made easy due to the availability of computer programs such as CHEV5L, developed by the Chevron Research Corporation (Ref 78) for layered analysis, and the Slab series, such as Slab 49 or Slab 17 (Ref 27), developed for slab analysis at the Center for Highway Research (CFHR), The University of Texas at Austin. Significant basic materials properties which are input into such programs include modulus of elasticity E and Poisson's ratio ν. Various procedures are currently in use for estimating the modulus of elasticity of asphalt mixtures. These include:

(1) static tests -
   (a) creep,
   (b) stress relaxation, and
   (c) constant rate of strain; and

(2) dynamic tests -
   (a) sinusoidal variations of stress or strain with time and
   (b) repeated loading-step function pulse loading; and

(3) indirect method - from charts such as those presented by van der Poel and modified by Heukelom and Klomp (Ref 21).

Each one of the above methods is important, and its use depends on various factors, such as the expected service condition of the asphalt mixture (e. g., parking lot pavements which are subjected to static loads and highway pavements which are subjected to dynamic loads) and the availability of test equipment, personnel, funds, time, etc.

In the case of highway pavements which are subjected to repeated traffic loads, it is generally felt that repeated loading, or dynamic, methods are more suitable than static tests.

Studies have indicated that fatigue life of asphalt concrete is dependent on its initial modulus. Thus, to better characterize asphalt materials for use in highway pavement, the manner in which the modulus of elasticity and Poisson's ratio vary during the entire life of the mixture should be defined and understood. This knowledge would permit the prediction of the elastic properties as a function of the number of load applications for use in pavement theories.

In laboratory fatigue testing, specimen failure has been defined by different criterion, ranging from crack initiation to complete fracture of the specimen. While crack initiation is difficult to observe and tends to underestimate fatigue life, complete fracture, on the other hand, usually gives an overly conservative estimate of fatigue life. Knowledge of the changes in elastic modulus and tensile properties as a result of load repetitions may help to explain the observed physical changes in the specimen and thus provide a better definition of its fatigue life (service life).

This study is intended to supply some of the needed information. An attempt was made to evaluate some of the factors affecting the indirect tensile fatigue life of asphalt mixtures and to evaluate the fatigue life predictive equations obtained from the indirect tensile test method. The effects of repeated loading on the modulus of elasticity, or resilient modulus, and Poisson's ratio were also investigated. Finally the changes in tensile strains and permanent strains which are distress manifestations of repeated loadings were examined.

The current status of knowledge pertaining to this study is presented in Chapter 2. Chapter 3 contains a comprehensive description of the experimental program. The results of the static tests are presented and discussed in Chapter 4. In Chapter 5 the fatigue test results, analyses, and discussions are presented while the effects of repeated loads on selected mixture properties such as resilient and permanent strains, resilient modulus, and Poisson's ratio are contained in Chapter 6. Finally, conclusions and recommendations based on the findings from this study are presented in Chapter 7.

CHAPTER 2. CURRENT STATUS OF KNOWLEDGE

Many studies on the fatigue behavior of asphalt-treated materials have been completed within the last two decades. This chapter summarizes the findings from the available literature concerning such studies.

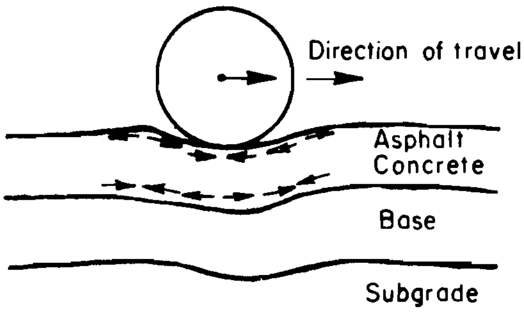
FATIGUE RELATED DEFINITIONS

Various terms have been used to describe fatigue behavior. An explanation or definition of each such term is presented below.

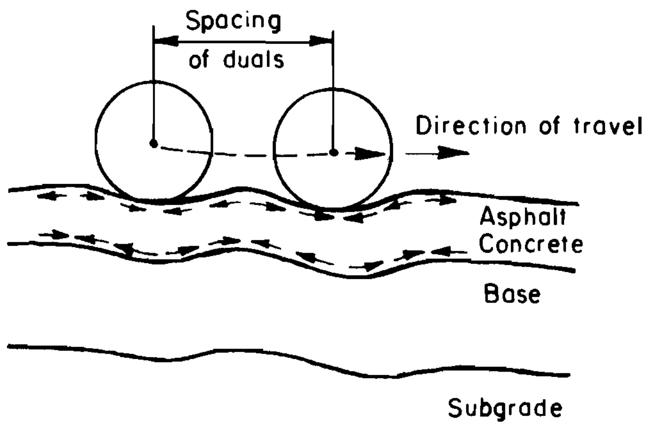
Fatigue has been defined as "a process of progressive localized permanent structural change occurring in a material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in cracks or complete fracture after a sufficient number of fluctuations" (Ref 3). It has also been defined as "a phenomenon of fracture under repeated or fluctuating stress having a maximum value less than the tensile strength of the material" (Ref 11). Figure 1 illustrates the fluctuating stresses and strains in an asphalt concrete pavement subjected to moving single-axle and tandem-axle loads. By bonding wire resistance strain gages to the surface of an asphalt concrete pavement, the strain-time relationship for a single-axle truck travelling at 4 and 20 mph was established by Monismith et al (Ref 51).

Failure, or fatigue life, has been described in terms of fracture life and service life. Fracture life N_f is the total number of load applications required to completely fracture a specimen. On the other hand, service life N_s is the total number of load applications necessary to cause the test specimen to perform at a level less than that originally intended (Fig 2).

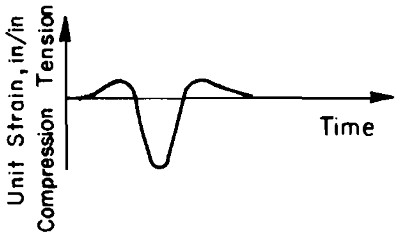
Mode of loading refers to the variation of stress and strain levels during a test. In a controlled-load, or controlled-stress, test, the applied load or stress level is kept constant until failure occurs. If the deflection or strain level is kept constant until failure occurs, the test is



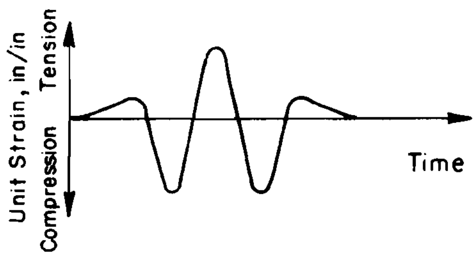
(a) Stresses in asphalt concrete in vicinity of single axle load.



(c) Stresses in asphalt concrete in vicinity of tandem axle load.

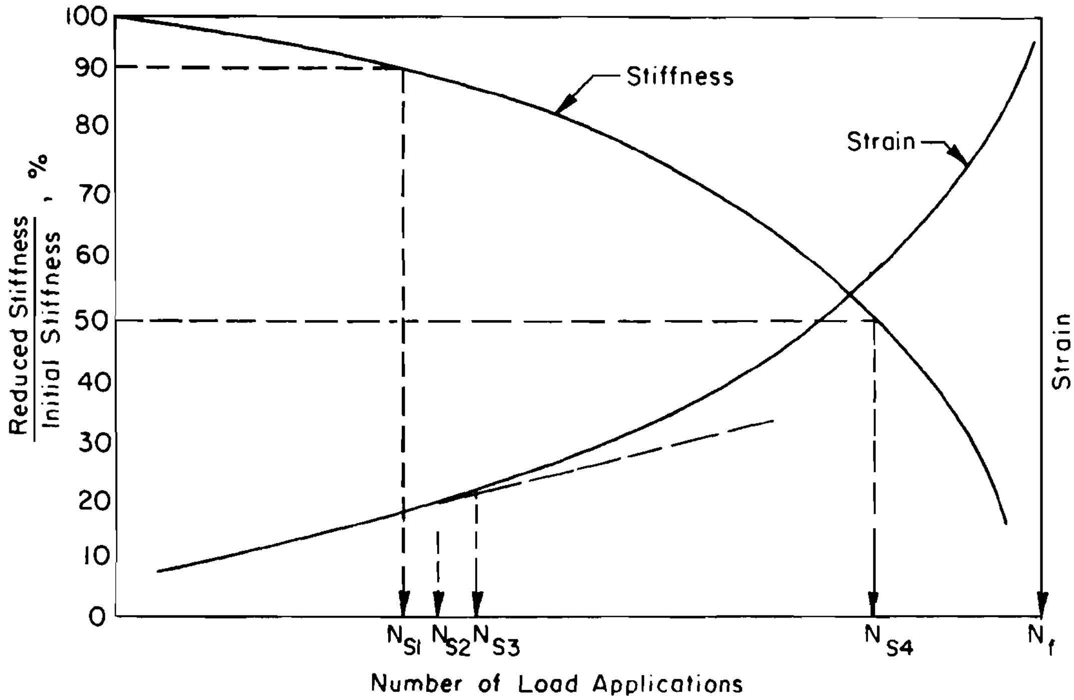


(b) Variation of strain with time at a point in surface of asphalt concrete due to moving single axle load.



(d) Variation of strain with time at a point in surface of asphalt concrete due to moving tandem axle load.

Fig 1. Variations of stresses and strains due to a moving load on an asphalt concrete pavement (Ref 51).



<u>Service Life</u>

N_{S1}: Number of load repetitions to cause a 10 percent reduction in initial stiffness.

N_{S2}: Number of load repetitions at which strain versus number of load applications curve deviates from linearity.

N_{S3}: Number of load repetitions to cause initiation of cracks (position varies).

N_{S4}: Number of load applications to cause a 50 percent reduction in initial stiffness.

<u>Fracture Life</u>

N_f: Number of load repetitions to cause a complete fracture.

Note: The positions of N_{S1}, N_{S2}, N_{S3}, and N_{S4} are not necessarily in the order shown.

Fig 2. Possible definitions of failure of a specimen subjected to laboratory fatigue testing.

called a <u>controlled-deflection</u>, or <u>controlled-strain</u>, test. A schematic representation of both modes of loading and a third, mixed or intermediate, mode is illustrated in Fig 3. Figure 3a illustrates that when the stress level is kept constant the strain increases with increasing load repetitions. For the controlled-strain mode shown in Fig 3c, the stress is decreasing with an increasing number of load applications, since the specimen is gradually damaged, requiring less load to produce the desired strain level.

<u>Mode factor</u> was introduced by Monismith and Deacon (Ref 47) in order to place the difference between controlled-stress and controlled-strain tests on a more quantitative basis and has been defined as

$$MF = \frac{/A/ - /B/}{/A/ + /B/} \tag{2.1}$$

where

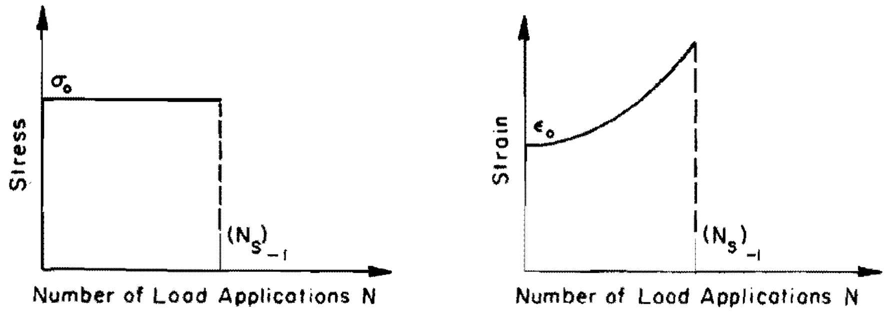
MF = mode factor,

/A/ = percentage change in stress due to an arbitrarily fixed reduction in stiffness,

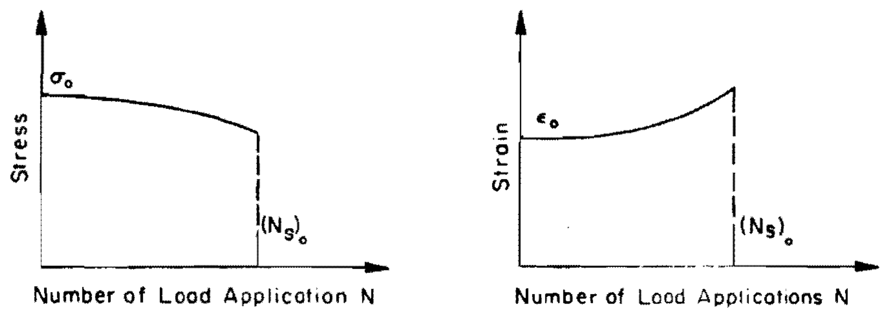
/B/ = percentage change in strain due to an arbitrarily fixed reduction in stiffness.

The mode factor equals -1 for controlled-stress loading, +1 for controlled-strain loading, and 0 or between -1 and +1 for the mixed mode. Monismith and Deacon (Ref 47) concluded from elastic layered analyses of a series of pavement sections that the controlled-strain mode of loading is applicable to thin flexible pavements (2 inches or less) while a controlled-stress test is applicable only to thick pavements (greater than 6 inches). A form of loading between the two modes is applicable to pavements between 2 and 6 inches thick.

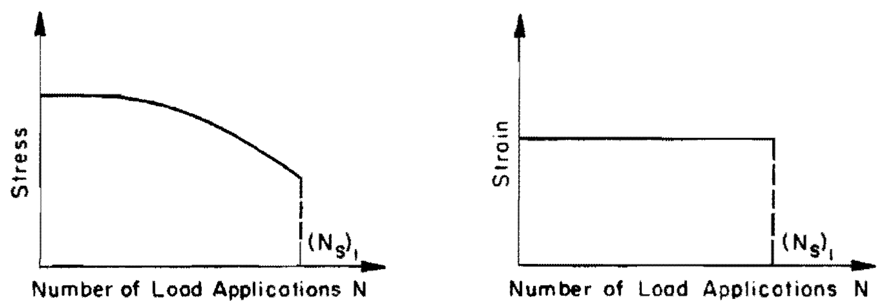
<u>Loading condition</u> is the set of values for load and environmental (e. g., temperature) variables during the testing period. A specimen is subjected to <u>simple loading</u> if the loading condition remains constant during the testing period. If the loading condition is changed during the test, the specimen is subjected to <u>compound loading</u>. Due to variations in both traffic-induced loads and environmental conditions, pavements actually are subjected to compound loading.



(a) Controlled-stress, mode factor = -1.



(b) Intermediate, mode factor = 0.



(c) Controlled-strain, mode factor =1.

Fig 3. Fatigue behavior of asphalt paving materials for various modes of loading (Ref 10).

Compound loading tests can be (1) sequence tests, in which different numbers of load applications N_1, N_2, N_3, etc. are applied at different levels of stress σ_1, σ_2, σ_3, etc., respectively, until failure occures; (2) repeated-block tests, in which a block of load applications is repeatedly applied until failure occurs, a block being comprised of 2 or more different numbers of applications at different stress levels, and the total number of applications within a block being termed the block size; and (3) random tests, in which the number of applications and the stress level are randomly applied until failure occurs. Other types of compound loading tests involve changes in temperature during a test. A test in which the number of load applications, the stress levels, test temperatures, and, if possible, moisture conditions are randomly applied during the test period would best simulate actual field conditions. However, such a "super-compound" loading test is also difficult to perform.

Cycle ratio, or percent cycles, R is the ratio of the number of load applications at any time n_i to the measured fatigue life of a specimen N. Thus,

$$R = \frac{n_i}{N}$$

in which R is expressed as a decimal, or as a percentage.

TESTING TECHNIQUES

Since actual pavements are subjected to fluctuating stresses and strains induced by traffic, the ultimate aim of any fatigue test, utilizing either laboratory prepared specimens or a model of an actual pavement, is to create stresses or strains (mostly tensile) using any one of a number of test methods, including the following:

(a) flexure or bending test,
(b) torsion test,
(c) direct tension test, and
(d) indirect tension test.

These test methods, together with some of the equipments that have been used, are described below.

Flexure Test

Tests in which beam specimens or plates are subjected to bending stresses by center-point loading, third-point loading, or cantilever loading are referred to as flexure or bending tests. The simplified equation utilized in such tests is of the form

$$\sigma = \frac{M}{Z} \tag{2.2}$$

where

σ = bending stress,

M = applied bending moment,

Z = section modulus of the specimen.

Figure 4 illustrates a point-loaded cantilever system used by Pell and Taylor to test cylindrical specimens of sheet asphalt under controlled-stress (Ref 62). One end of the specimen was clamped to a rotating shaft while the other was attached to a weight, thereby applying a bending moment to the clamped end. The specimen was made with the cross section at the center reduced to insure that failure occurred away from the clamped end.

Equipment was used by Jimenez and Gallaway (Refs 29 and 30) in which circular plate specimens were loaded by means of eccentric rotating weights and support was provided by a pressurized fluid. The test approached a controlled-stress test since the deflection of the specimen generally increased with increased load repetitions.

Monismith (Ref 43) utilized equipment in which a beam specimen rested on a spring base to simulate field support conditions and was subjected to controlled-stress or controlled-strain loading. In the controlled-stress test, a fixed load was pneumatically applied to the specimen through the loading system, while, in the controlled-strain test, wire strain gages were bonded to the underside of the beam and the load was controlled to produce a fixed level of strain in the gages.

Equipment developed by Deacon (Ref 6) applied both simple and compound loading to specimens (Fig 5). The equipment applied a constant moment to

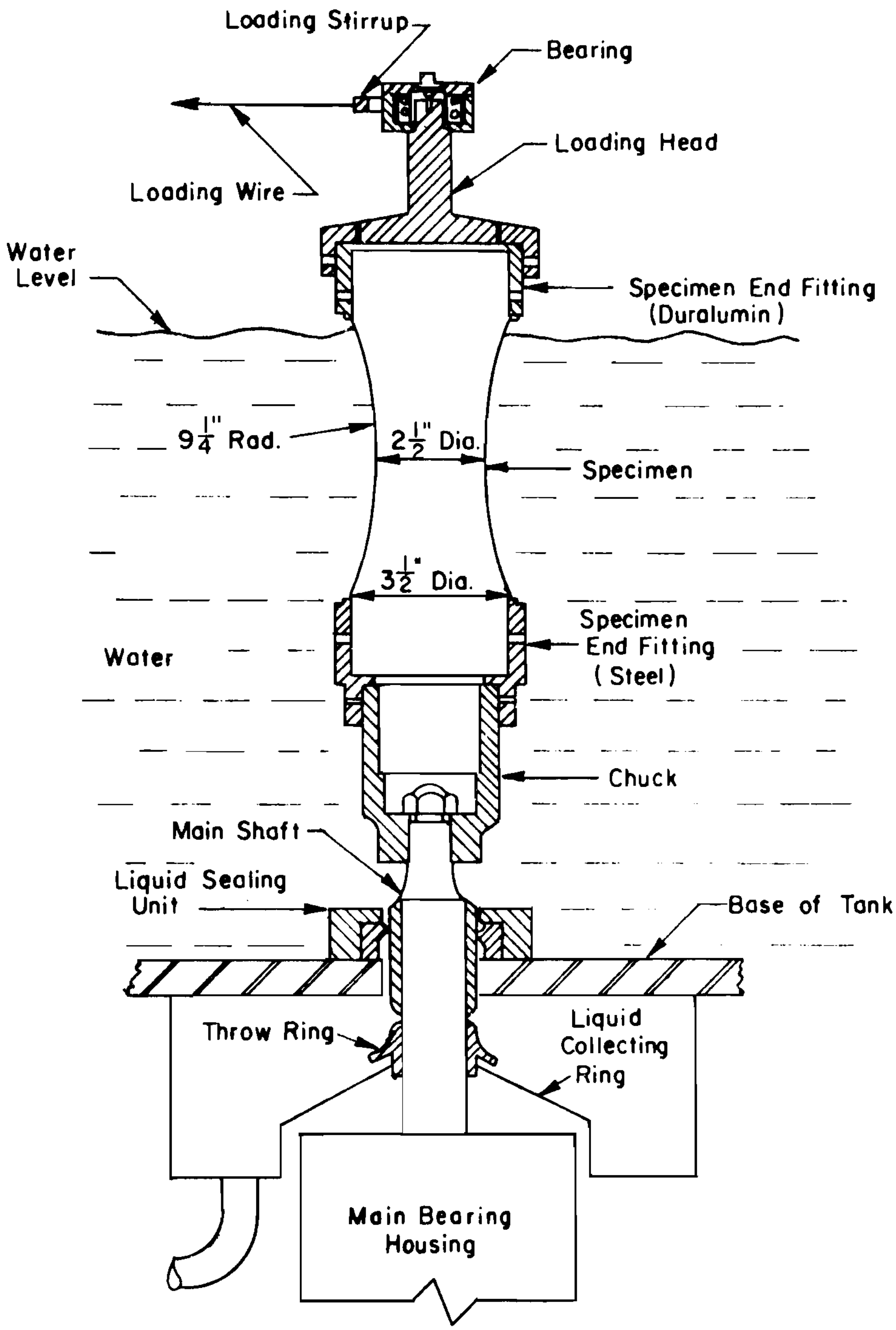
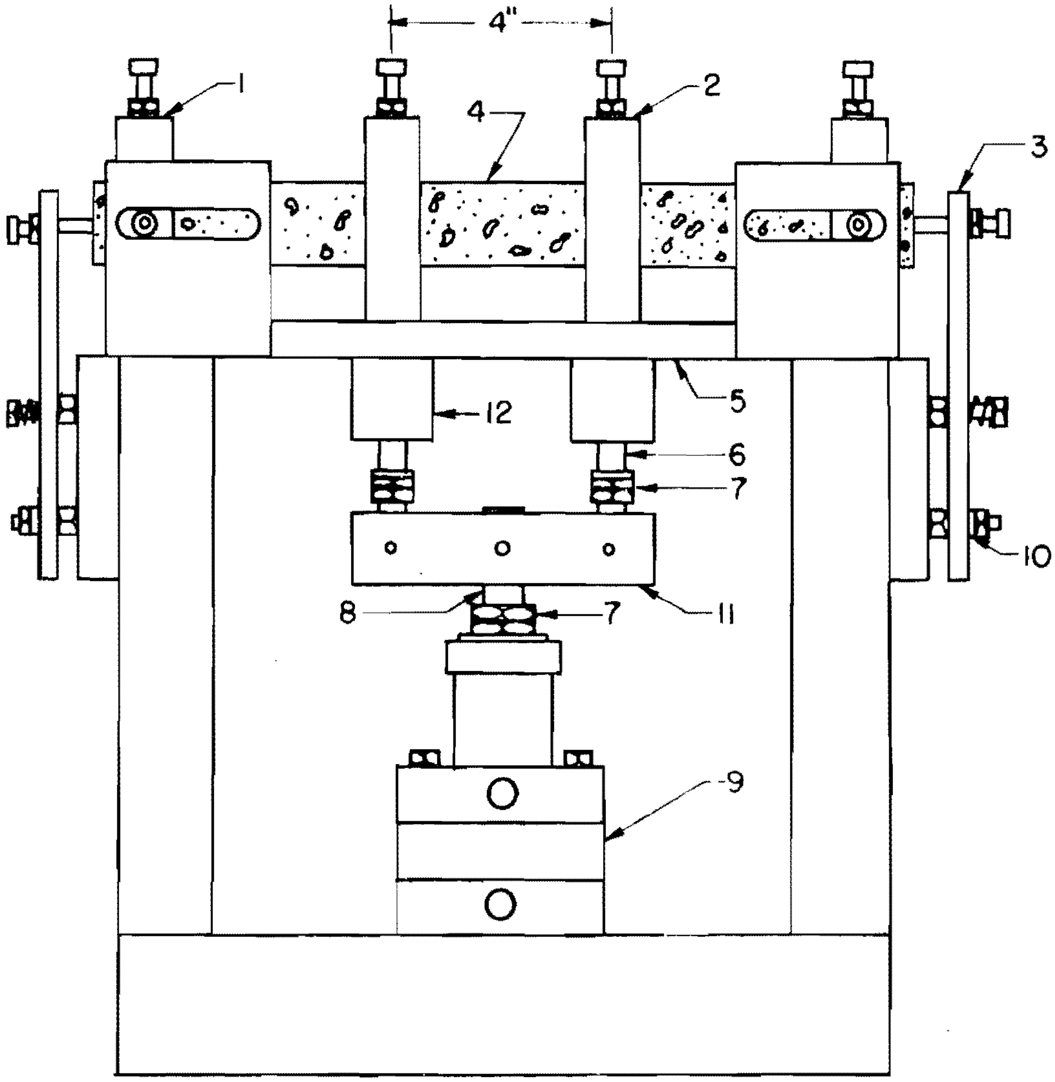


Fig 4. Detail of specimen and fatigue loading system (Ref 62).



Key:

1. Reaction clamp
2. Load clamp
3. Restrainer
4. Specimen
5. Base plate
6. Loading rod
7. Stop nut
8. Piston rod
9. Double-acting, Bellofram cylinder
10. Rubber washer
11. Load bar
12. Thomson ball bushing

Fig 5. Repeated flexure apparatus (Ref 6).

the center of the beam specimen and was capable of being used in both controlled-stress and controlled-strain tests.

Torsion Test

In torsion tests, specimens are subjected to torsional strains by the application of a torque or twisting moment. In such cases, the expression below applies:

$$\theta = \frac{T}{GJ} \tag{2.3}$$

where

θ = angle of twist, measure of torsional strain,
T = applied torque,
G = shear modulus of material,
J = polar moment of inertia (also termed torsional constant), and
GJ = torsional rigidity.

Pell (Refs 57 and 58) applied controlled torsional strain to specimens using the equipment shown in Fig 6.

Direct Tension Test

Tests in which a specimen is subjected to direct tensile stresses by pulling in opposite directions are direct tension tests. The stress in this case is obtained from the simple expression

$$\sigma = \frac{P}{A} \tag{2.4}$$

where

σ = stress induced,
P = applied load, and
A = cross-sectional area of specimen.

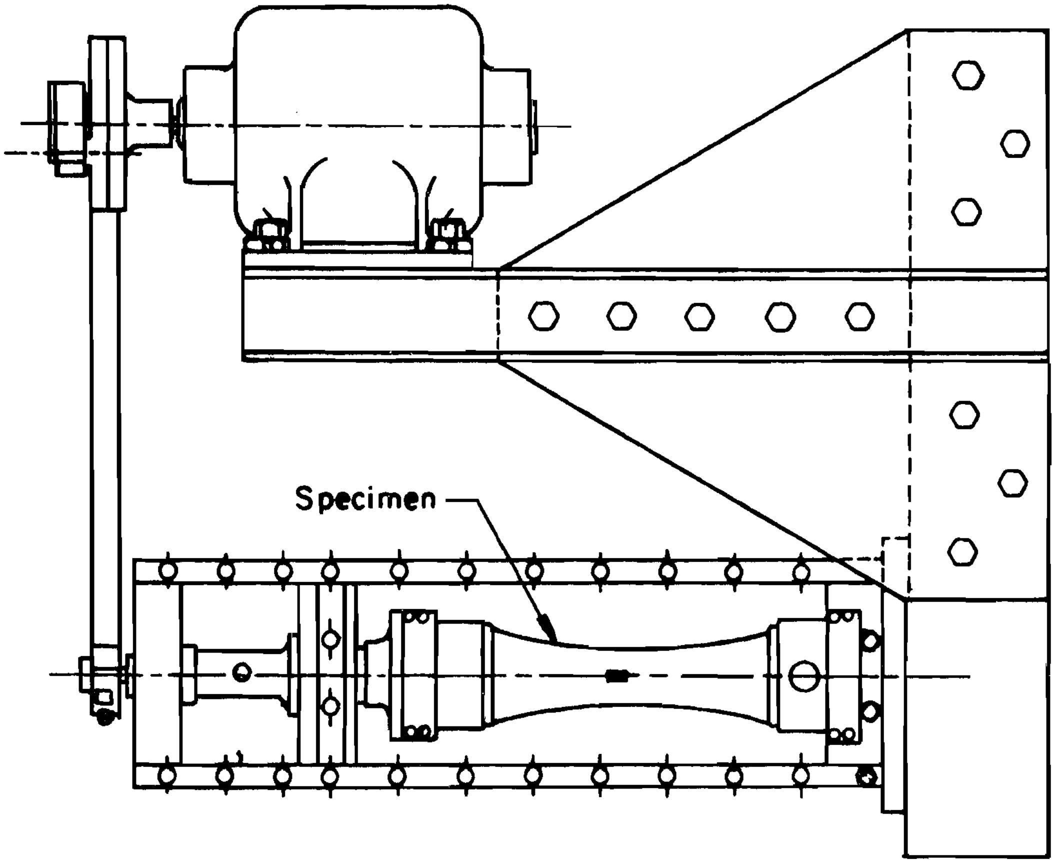


Fig 6. Controlled torsional strain fatigue machine (Ref 58).

Even though the test is simple in theory, the problems of end gripping and stress concentration have restricted the use of this method. Figure 7 illustrates the direct tension test utilized by Epps (Ref 8).

Indirect Tension Test

The indirect tension, or splitting tension, tests are those tests in which cylindrical specimens are subjected to compressive loads distributed along two opposite generators, a condition which creates a relatively uniform tensile stress perpendicular to and along the diametral plane which contains the applied load. A detailed review of this test method including its theory is contained in Refs 16 and 26.

Other Tests

Wood and Goetz (Refs 79 and 80) and Goetz et al (Ref 14) have utilized both the unconfined and confined compression tests to investigate the effects of repeated loads on specimens of sheet-asphalt mixtures. Similarly, Raithby and Sterling have investigated the fatigue resistance of hot-rolled asphalt under reversed axial loading, using the tension/compression test (Refs 64 and 65).

TYPICAL FATIGUE TEST RESULTS

From the number of studies thus completed on fatigue behavior of asphalt mixtures, it can be said that there is consensus regarding the variational characteristics of fatigue life and its relationships with stress and strain. A summary of these characteristics and relationships is presented in this section.

Variational Characteristics

It has been found that apparently identical specimens of asphalt mixtures subjected to fatigue loading under exactly the same conditions have produced different fatigue lives. This difference, which can be larger than one order of magnitude, is partially due to the difficulty of producing exactly identical specimens and the inherent errors in fatigue testing.

The magnitude of the variations in fatigue life is often presented in the form of standard deviation or coefficient of variation CV , a ratio of

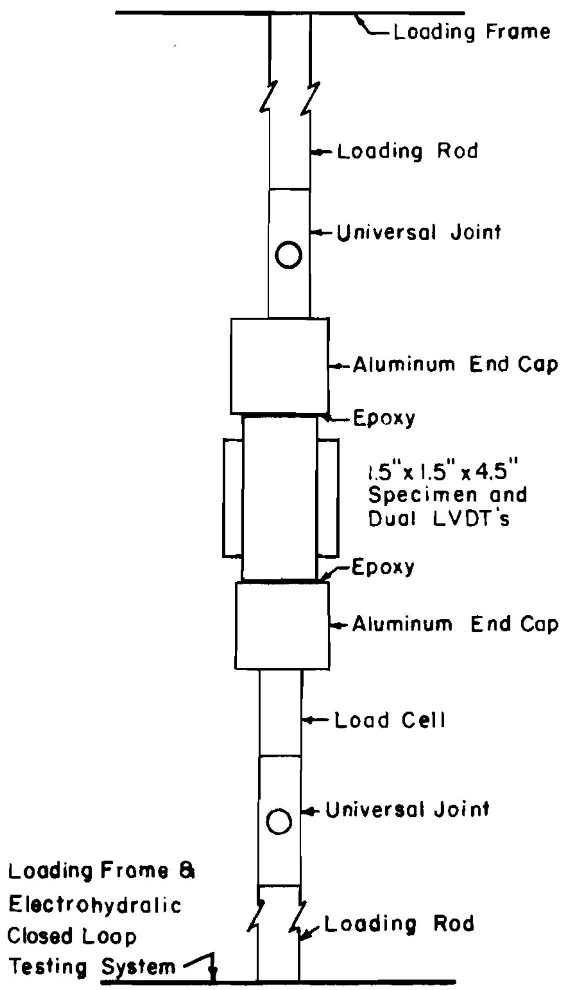


Fig 7. Direct tension testing apparatus (Ref 8).

standard deviation to the mean fatigue life. Values of coefficient of variation, including those for field specimens and laboratory prepared specimens, have been customarily high.

For laboratory specimens, Finn (Ref 11) summarized CV values which ranged from 54 to 76 percent. Similar values have also been reported by a number of investigators, including Vallerga et al (Ref 75), Moore and Kennedy (Ref 52), and Monismith et al (Ref 48).

For field specimens, the CV values are also generally high. Monismith et al (Ref 48) reported values which ranged between 25 and 131 percent. Similarly, Navarro and Kennedy (Ref 55) have reported CV values which ranged between 26 and 84 percent.

Most test statistics are based on the assumption of a normally distributed population. In the case of fatigue tests, Pell and Taylor (Ref 62) and Kasianchuk (Ref 33) have reported that fatigue life has a logarithmic normal distribution. Figure 8 illustrates the distribution obtained by Pell and Taylor for 100 test specimens.

Relationship with Stress and Strain

Fatigue life is often related to applied repeated stress and estimated initial strain in stress-controlled tests and to applied repeated strain in strain-controlled tests. The time-consuming nature of fatigue testing has given rise to the development of a short cut with which fatigue life can be estimated by the use of these relationships. Data from various studies indicate that both the tensile stress and tensile strain are good fatigue life determinants and that an approximately linear relationship exists between the logarithm of stress or strain and the logarithm of fatigue life.

Applied Stress. The general stress-fatigue life equation is of the form

$$N = K_2 \left(\frac{1}{\sigma}\right)^{n_2} \tag{2.5}$$

where

N = number of load applications to failure (N_f or N_s),

σ = applied repeated stress,

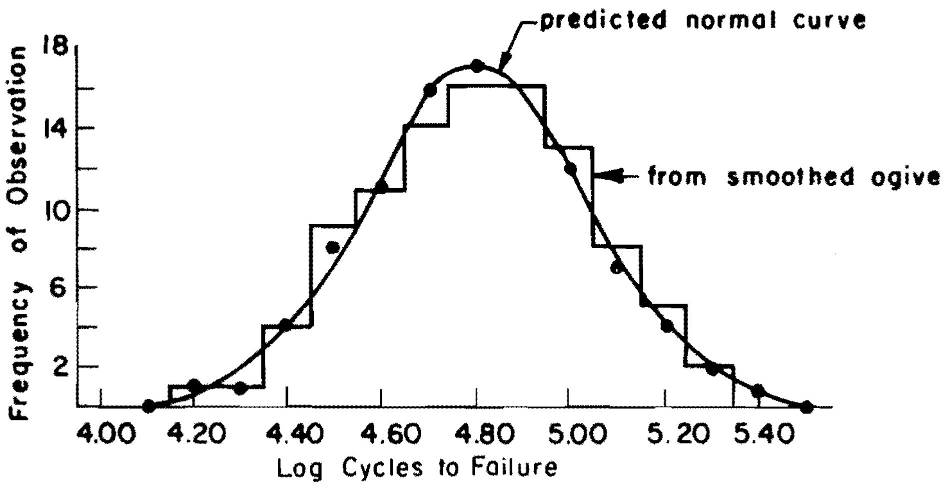


Fig 8. Histogram of 100 fatigue test results (Ref 62).

K_2 = antilog of the intercept of the logarithmic relationship between stress and fatigue life, and

n_2 = slope of the logarithmic relationship between stress and fatigue life.

In a recent study, Porter and Kennedy (Ref 63) suggested that Eq 2.5 be based on a stress difference in the case of loading conditions which involve a biaxial state of stress. Thus, Eq 2.5 becomes

$$N = K_2' \left(\frac{1}{\Delta\sigma}\right)^{n_2} \tag{2.6}$$

where

K_2' = antilog of the intercept of the logarithmic relationship between stress difference and fatigue life,

$\Delta\sigma$ = stress difference, or the algebriac difference between the horizontal and vertical stresses, and

N and n_2 are as defined above.

Typical values of K_2, K_2', and n_2 obtained from controlled-stress flexural, axial load, and indirect tensile tests on various mixtures are summarized in Appendix A. From the summary, K_2 and K_2' generally ranged from about 2.5×10^6 to 8.2×10^{21} while n_2 ranged from about 1.58 to 7.1.

Initial or Applied Strain. A linear relationship is generally found to exist between the logarithm of fatigue life and the logarithm of initial strain (stress-controlled test) or the logarithm of applied repeated strain (strain-controlled test). Pell et al (Ref 61) have shown that tensile strain is the prime determinant of fatigue life and that, for constant-strain tests, fatigue life is independent of temperature and rate of loading effects. The general strain-fatigue life equation is of the form

$$N = K_1 \left(\frac{1}{\varepsilon_{mix}}\right)^{n_1} \tag{2.7}$$

where

N = number of load applications to failure (N_f or N_s),

ε_{mix} = applied strain or initial strain in the mixture,

K_1 = intercept of the regression line, and

n_1 = slope of the regression line.

The values of K_1 and n_1 obtained from controlled-stress flexural and axial load tests are summarized in Appendix A. Values of K_1 generally ranged from 5.0×10^{-20} to 6.5×10^{-5} while n_1 ranged from 1.2 to 6.3.

Pell and Cooper (Ref 60) reported that a relationship exists between the constants K_1 and n_1 as follows:

$$n_1 = 0.5 - 0.313 \log K_1 \tag{2.8}$$

A relationship between n_1 and K_1 may simplify the general strain-fatigue life relationships (Eq 2.7) by reducing the number of constants in the equation.

In order to eliminate the effect of loading rate, temperature, and asphalt content, Pell et al (Ref 61) and Pell (Ref 58) suggested that fatigue life actually depends on strain in the bitumen per se and that the following expression applies:

$$N = K_1 \left(\frac{1}{\varepsilon_{bit}} \right)^{n_1} \tag{2.9}$$

where

ε_{bit} = strain in bitumen and

N, K_1, and n_1 are as previously defined.

ε_{bit} can be computed from the relationship

$$\varepsilon_{bit} = \frac{\varepsilon_{mix}}{K_o B_v} \tag{2.10}$$

where

K_o = a constant depending on type of aggregate and amount of filler or voids in the mix, and

B_v = volume concentration of bitumen

$$= \frac{\text{volume of bitumen}}{\text{volume of aggregate + volume of bitumen}}$$

Stress-Strength Ratio. Moore and Kennedy (Refs 52 and 53) found a linear relationship between the logarithm of fatigue life and the logarithm of stress-strength ratio, a ratio of repeated tensile stress σ_T to the estimated tensile strength S_T of the material (Fig 9). This sounds reasonable in that division of stress by tensile strength might eliminate the effects of asphalt content and temperature on the stress-fatigue life relationships. Navarro and Kennedy (Ref 55), however, did not find a strong relationship for field cores of asphalt concrete and blackbase, possibly due to the large amount of variation associated with field cores. Figure 9 shows that the regression line obtained by Navarro and Kennedy fell fairly well within the data presented by Moore and Kennedy.

FACTORS AFFECTING FATIGUE CHARACTERISTICS OF ASPHALT MIXTURES

Many factors have been found to affect the fatigue behavior of asphalt mixtures. Deacon and Monismith (Ref 7) have summarized these factors under the broad categories of load, mixture and specimen, and environmental variables. The effects of these variables on fatigue life and stiffness of asphalt concrete mixtures have also been summarized by Monismith et al (Ref 48) as presented in Table 1.

Load Variables

A large number of variables affecting the fatigue behavior of asphalt concrete have been associated with loading conditions. Some of these are magnitude, mode, frequency, duration, and rest period.

Load Magnitude. All tests have indicated a longer fatigue life for the smaller loads (stresses or strains). This has been reported in the form of stress-fatigue life and strain-fatigue life relationships, which are generally linear on log-log plots, as discussed earlier in this chapter.

Mode of Loading. The difference in behavior of materials subjected to stress-controlled and strain-controlled tests was discussed by Monismith (Refs 10, 45, and 46). As illustrated in Fig 10, a longer fatigue life is obtained with materials tested under controlled-strain mode. This has been explained in terms of the energy input (stress X strain), which is larger for the controlled-stress tests.

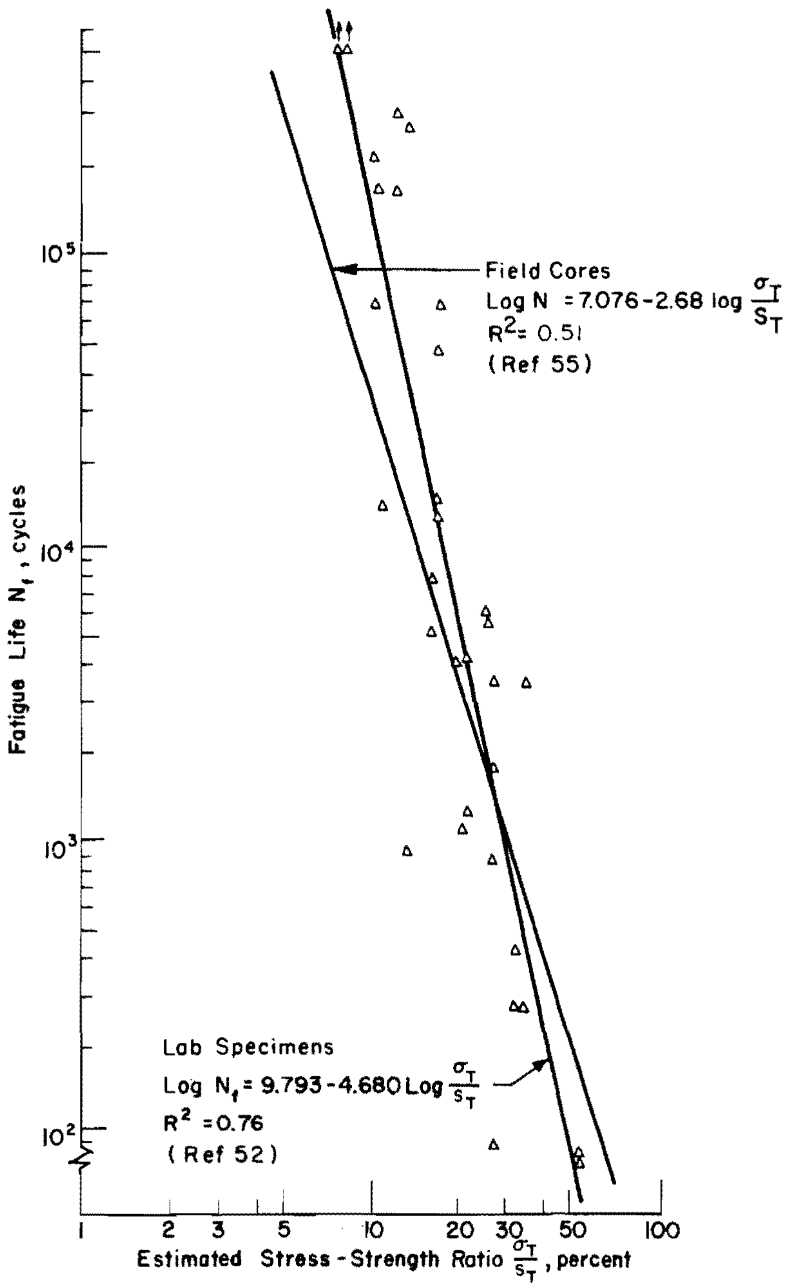


Fig 9. Relationship between logarithm of fatigue life and logarithm of stress-strength ratio.

TABLE 1. FACTORS AFFECTING THE STIFFNESS AND FATIGUE BEHAVIOR OF ASPHALT CONCRETE MIXTURES (REF 48)

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

[a]Reaches optimum at level above that required by stability considerations.

[b]No significant amount of data; conflicting conditions of increase in stiffness and reduction of strain in asphalt make this speculative.

[c]Approaches upper limit at temperature below freezing.

[d]No significant amount of data.

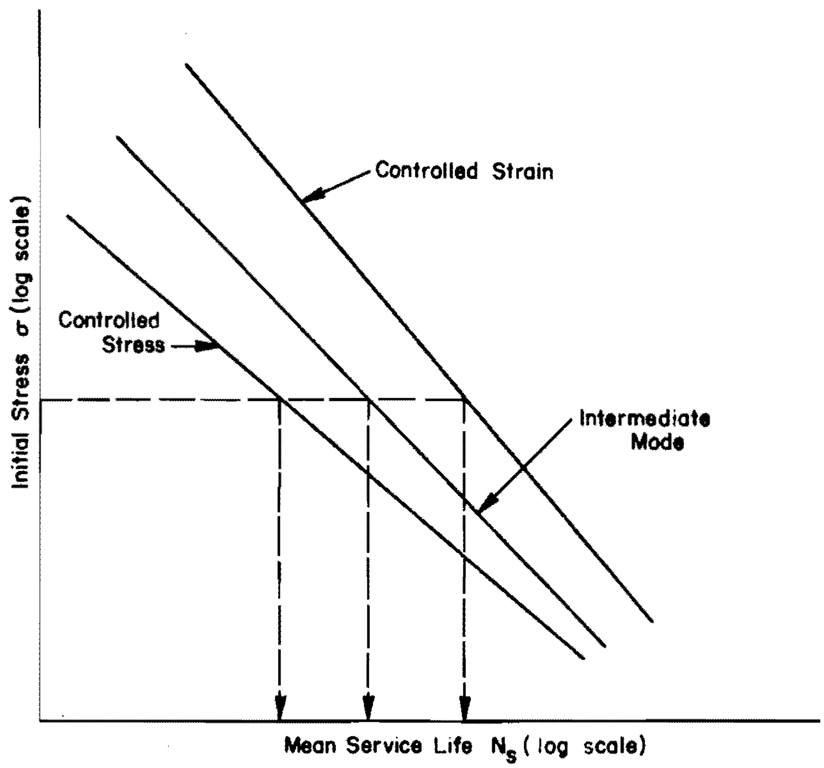


Fig 10. Hypothetical fatigue diagrams illustrating effect of mode of loading (Ref 10).

Loading Frequency, Duration, and Rest Period. The effects of loading frequency, duration, and rest period are interdependent and a change in any one of these factors will affect at least one other factor.

Monismith, Secor, and Blackmer (Ref 51) have evaluated the effect of loading frequency based on the relative damage induced by repetitive loading expressed as a ratio of the modulus of rupture at certain repetitions to the modulus of rupture of specimen in an unflexed condition. Utilizing the results from frequencies of 3, 15, and 30 applications per minute, they concluded that there was no apparent effect produced by rate of load application. However, in later studies, Pell (Ref 59) and Pell and Taylor (Ref 62) reported a significant effect of frequency, with higher frequencies producing a longer fatigue life. Based on studies utilizing a controlled-stress mode of loading on dense graded asphalt mixtures at 75°F, Deacon and Monismith (Ref 7) reported that increasing the frequency of load application in the range of 30 to 100 repetitions per minute significantly decreased fatigue life, by approximately 20 percent.

Raithby and Sterling (Ref 64) have investigated the effect of a rest period by subjecting prismatic specimens to direct tensile and compressive loading. They presented results which suggest that for short rest periods, fatigue life increases rapidly with increasing rest periods "and then appears to reach a limiting value at about 400 ms, beyond which increasing the duration of the rest period has very little further effect." In an earlier study (Ref 57), Pell utilized both controlled-stress and controlled-strain tests to investigate the effects of prolonged rest periods on fatigue lives of sandsheet specimens. This was done by loading a specimen for one-fifth to one-half of its mean life and then allowing a rest period of between one hour and 24 hours before the specimen was again loaded, until failure. For the temperature range of 32 to 104°F, Pell concluded that there was no beneficial effect of a rest period.

Deacon reported that increasing load durations produced shorter fatigue lives (Ref 6). This was done by keeping the frequency constant, which in effect decreased the rest period.

Mixture Variables

These include such variables as aggregate type and gradation, asphalt type, and asphalt content. The manner in which these variables affect fatigue life of asphalt concrete mixtures depends very much on the mode of loading.

Aggregate Type. Various studies have shown that in a controlled-stress test aggregate type has a negligible effect on fatigue life (Refs 9, 35, and 59). However, Jimenez and Gallaway (Ref 29) have shown that aggregate type is important in that it determines the amount of asphalt required in the mixture. Increased roughness and angularity, a condition which requires a larger amount of asphalt, may produce increased fatigue life in a stress-controlled test or a decrease in fatigue life in a strain-controlled test.

Aggregate Gradation. Monismith (Ref 45) reported that for comparable asphalt contents dense graded mixtures exhibit longer fatigue lives than those with open graded aggregates in controlled-stress tests. Epps and Monismith (Ref 9) examined data from various studies (Refs 4, 35, and 59) and concluded that the effects of aggregate gradation could be explained by differences in the asphalt and air void contents.

Both Pell (Ref 59) and Taylor (Ref 73) demonstrated the concept of an optimum filler content for maximum fatigue life.

While Taylor (Ref 73) found that the type and grading of coarse aggregates did not greatly influence the fatigue life, Epps (Ref 8) showed that finer gradings produced mixes with longer fatigue lives but that a greater compactive effort was required.

In controlled-strain tests, Kirk (Ref 35) suggested that the effect of gradation is negligible even though limited data seemed to indicate a decrease in fatigue life when gradation is changed from open to dense (Table 2).

Asphalt Penetration. Pell (Ref 59) conducted controlled-stress fatigue tests on two grades of asphalt cement, 40 to 50 pen and 90 to 100 pen, and concluded that the grade of bitumen appears to affect the fatigue life mainly through its influence on the slope of the fatigue line.

Jimenez (Ref 28), Bazin and Saunier (Ref 4), Vallerga et al (Ref 75), Monismith et al (Ref 51), and Moore and Kennedy (Ref 52) have all indicated that fatigue life increases with an increase in asphalt viscosity, i.e., a decrease in penetration. Epps and Monismith (Ref 9) have similarly shown that longer fatigue lives are obtained for mixtures with higher stiffness (or lower asphalt penetration).

From controlled-strain test results, Kirk (Ref 35) found that the fatigue strain at 1,000,000 load applications is independent of asphalt viscosity for penetrations ranging from 50 to 200. However, Santucci and Schmidt (Ref 68) indicated that softer asphalts produce longer fatigue lives.

Asphalt Content. In general, it has been found that there is an optimum asphalt content for maximum fatigue life (Fig 11). This has been found to be true for controlled-stress tests by Pell (Ref 59), Epps (Ref 8), Epps and Monismith (Ref 9), and Pell and Taylor (Ref 62). Jimenez and Gallaway (Ref 29) have also indicated that there is an optimum asphalt content for maximum fatigue life, which is a function of aggregate type. Epps (Ref 8) has similarly shown that the optimum asphalt content based on the fatigue behavior of basalt aggregate mixture was 6.7 percent, which was 0.8 percent above the asphalt content required for stability.

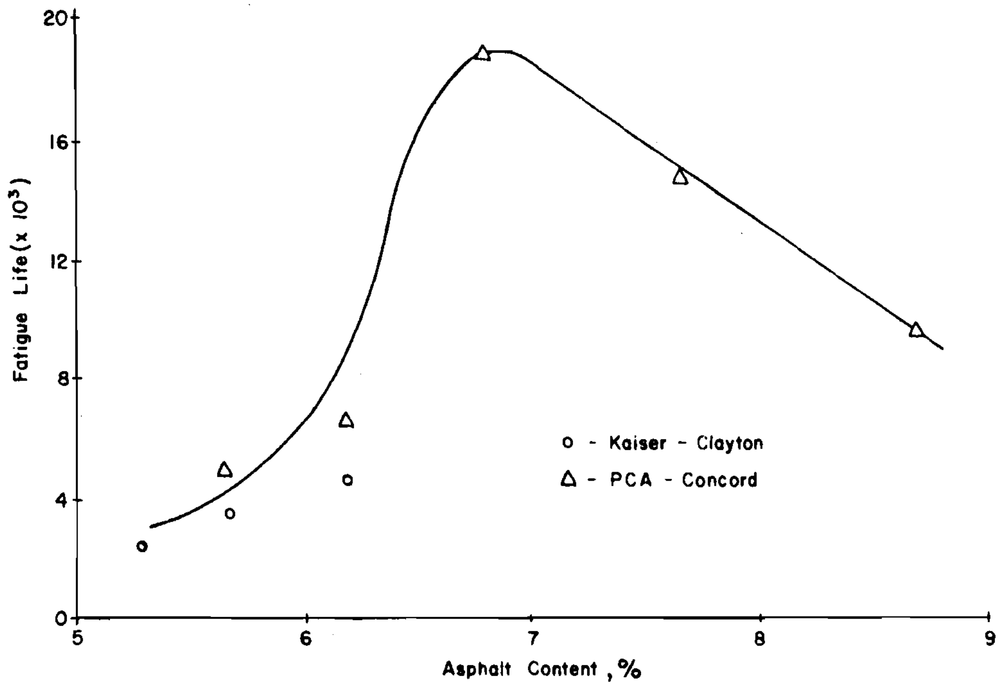
Environmental Variables

Variables such as temperature, moisture, and any other factor altering material properties during service life are referred to as environmental variables. Results of studies concerned with their effects are summarized below.

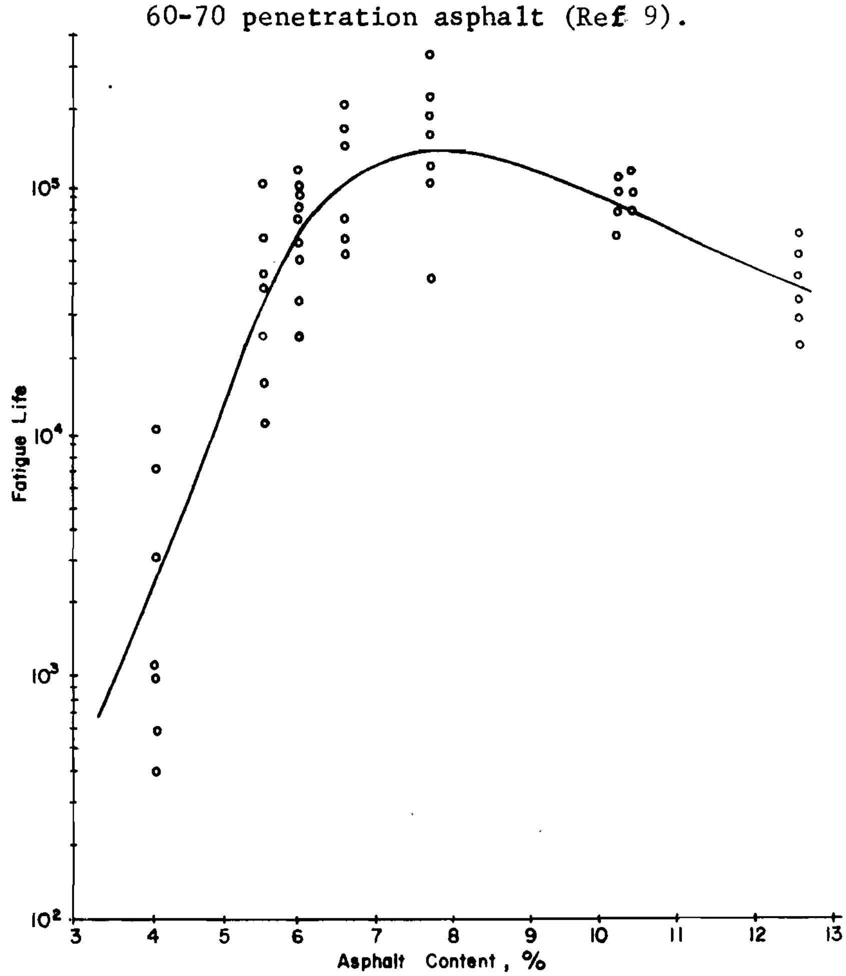
Testing Temperature. In a controlled-stress test, a decrease in testing temperature results in an increase in fatigue life (Fig 12a). This has been shown by data presented by Saal and Pell (Refs 66 and 67), Jimenez and Gallaway (Ref 30), Epps (Ref 8), and Taylor (Ref 73). In addition, Pell and Taylor (Ref 62) have stated that the influence of temperature within limits can be explained in terms of specimen stiffness.

In controlled-strain tests, the fatigue life increases with an increase in temperature (Fig 12b) as demonstrated by data presented by Pell (Ref 57), Monismith (Ref 44), and Garrison (Ref 13). This effect has been attributed to the slower rate of crack propagation at high temperatures since the load or stress is reduced as the specimen is damaged.

Moisture. The effects of moisture on the fatigue life of asphalt concrete mixtures have not been directly reported. However, since moisture reduces resilient modulus (Ref 70) and tensile strength (Ref 36) of asphalt mixtures, it will probably reduce its fatigue life.

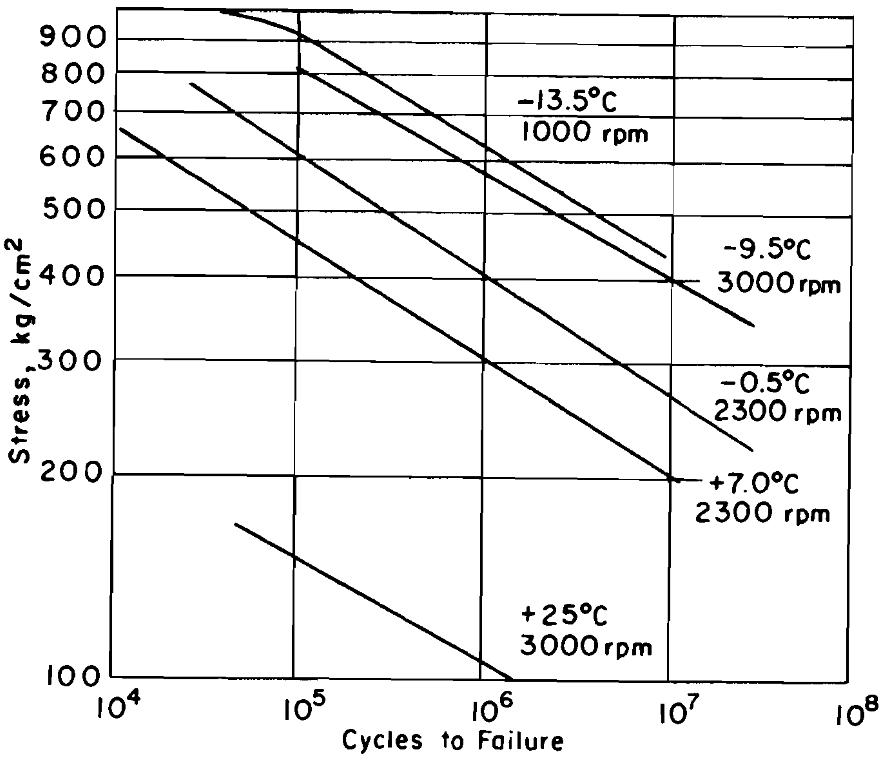


(a) California medium grading, basalt aggregate, 60-70 penetration asphalt (Ref 9).

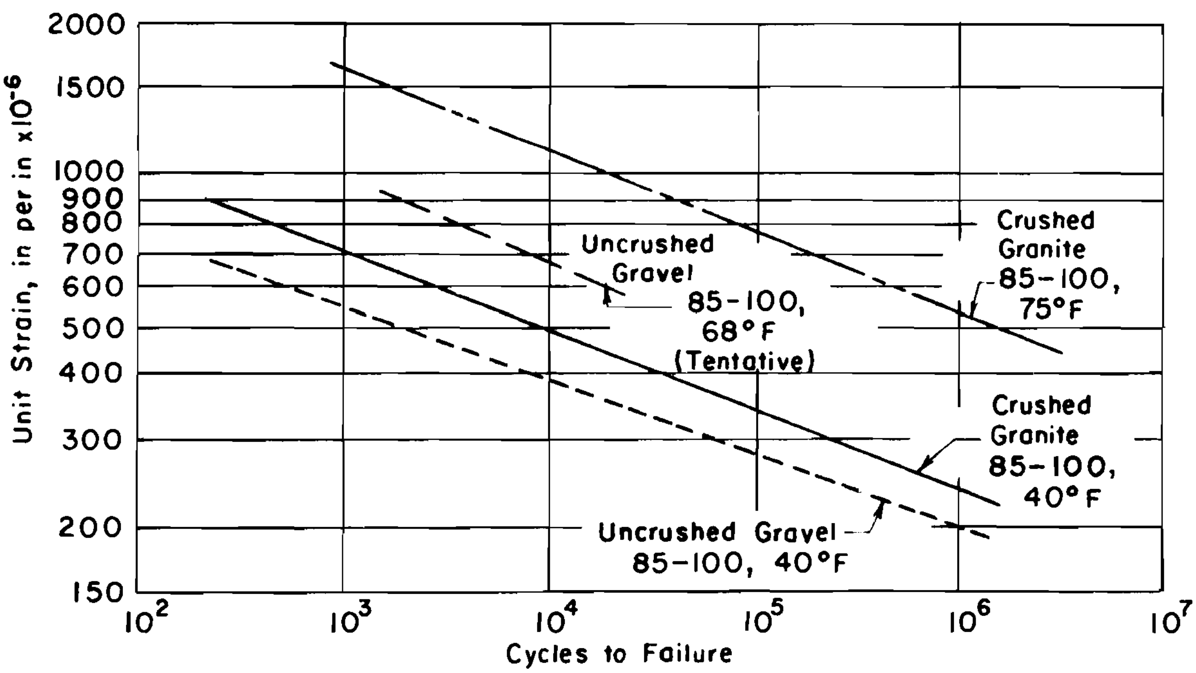


(b) Crushed rock, base course mixture, 40-50 penetration bitumen (Ref 62).

Fig 11. Effect of asphalt content on fatigue life.



(a) Constant bending stress (Ref 66).



(b) Constant strain (Ref 44).

Fig 12. Influence of temperature on results of constant-stress and constant-strain amplitude fatigue tests.

Other Variables

Efforts have been made to explain fatigue behavior in terms of other specimen properties, such as stiffness and air void content. Results from previous studies are summarized below.

Stiffness or Modulus. Deacon and Monismith (Ref 7) concluded that the stiffness of a mixture is important to its fatigue characteristics and that any factor affecting stiffness will similarly affect its fatigue behavior. Generally in the controlled-stress test, fatigue life increases as the mixture stiffness increases (Fig 13).

In a controlled-strain test, an increase in stiffness produces a decrease in fatigue life. Santucci and Schmidt (Ref 68) have presented data which support the above and which indicate that measured service life for a particular mix can be predicted from initial stiffness values.

Air Void Content. Results of controlled-stress tests by Saal and Pell (Ref 67), Bazin and Saunier (Ref 4), Epps (Ref 8), Pell and Taylor (Ref 62), and Epps and Monismith (Ref 9) have shown that mixes containing high air void contents have shorter fatigue lives than those with low air void contents (Fig 14). Epps (Ref 8), however, has suggested that in addition to the absolute volume of the voids, the size, shape, and degree of interconnection are also important. Thus, the presence of one large void will reduce the load-carrying solid cross section of the specimen more than several smaller voids scattered throughout the specimen.

In controlled-strain tests, Santucci and Schmidt (Ref 68) have also shown that reduced air void contents lead to longer fatigue lives.

EFFECTS OF REPEATED LOADING

Studies have indicated that materials behave differently under repetitive loading than they do under static loading. The behavior of mixes under repeated load is complex, depending on such variables as stress level, amount and type of asphalt, and temperature. It is thus inadequate to apply static test data to dynamic loading conditions in predicting the tensile and elastic properties of asphalt mixtures.

Despite many studies concerning the fatigue behavior of asphalt mixtures, the repeated loading effects on properties such as elastic modulus,

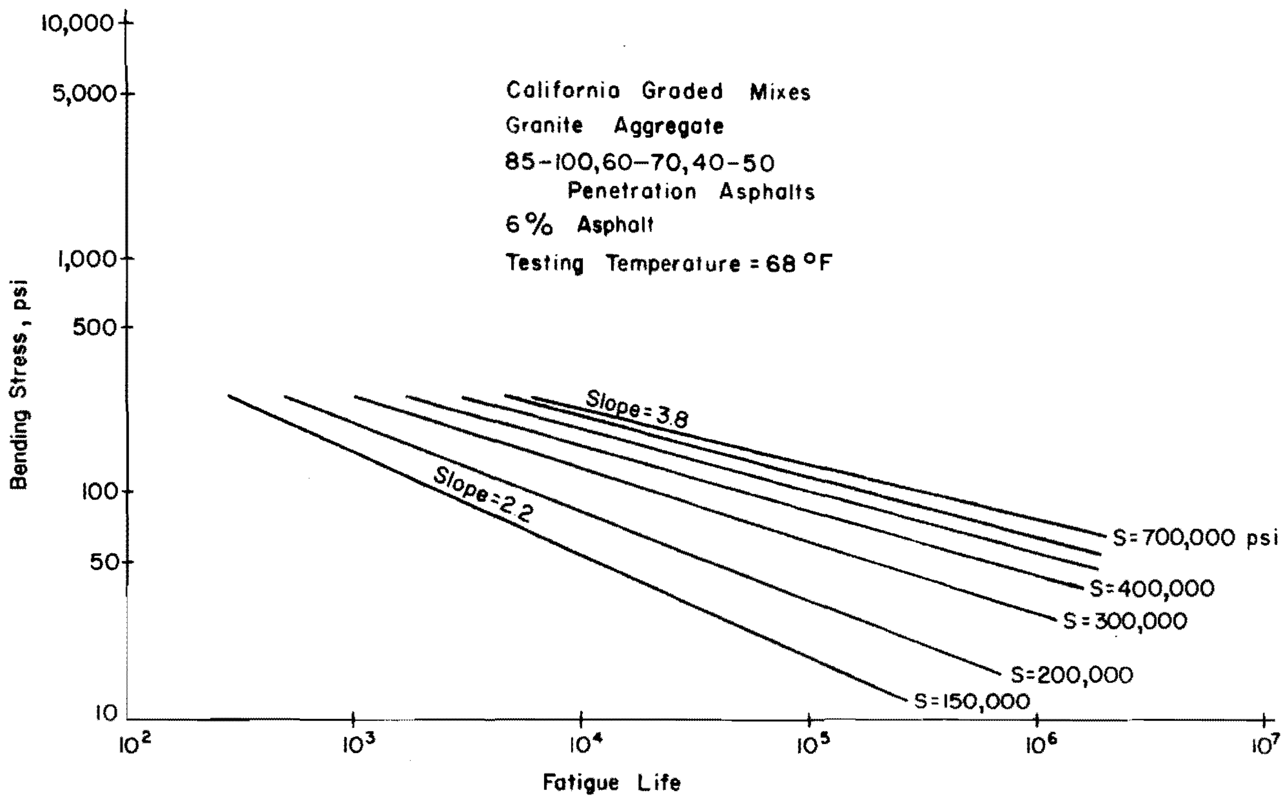
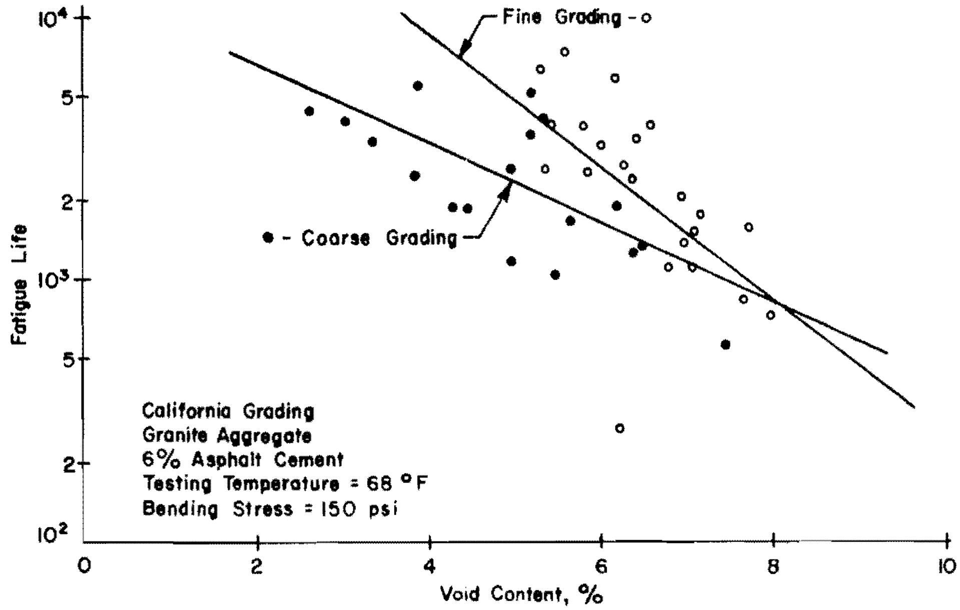
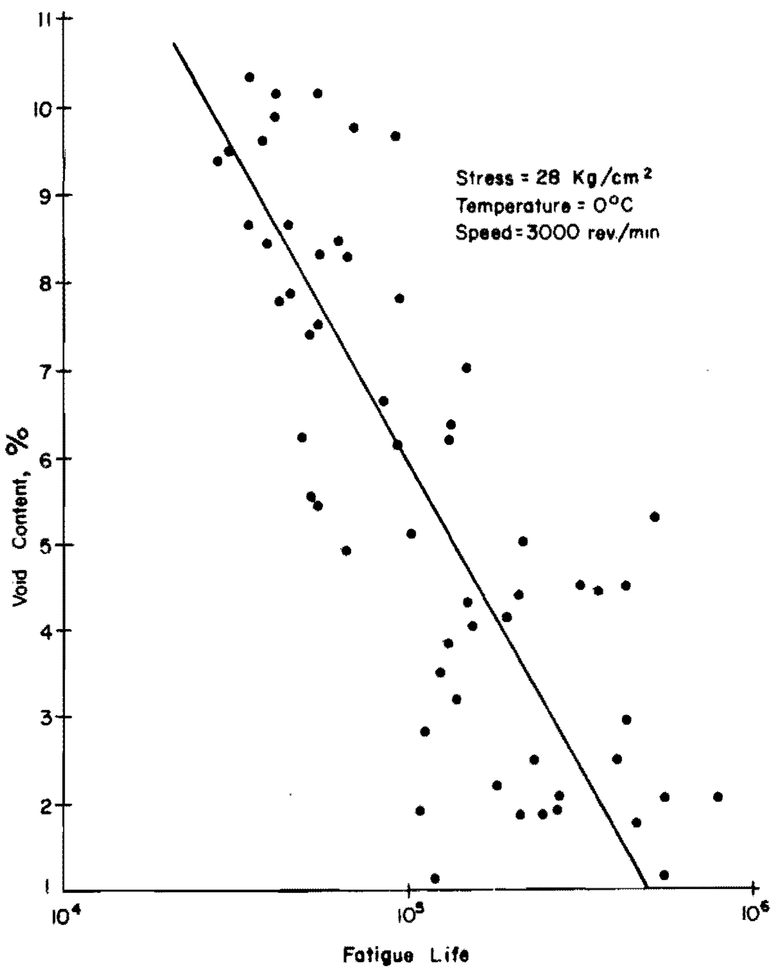


Fig 13. Bending stress versus applications to failure, for mixtures of different stiffness (Ref 9).



(a) From Ref 67.



(b) From Ref 9.

Fig 14. Effects of air void content on fatigue life.

Poisson's ratio, tensile strain, and tensile strength, which affect the design and performance of asphalt concrete pavements, have not been well documented. This may be due to the difficulty in developing standard test procedures and the fact that a pavement design method utilizing such information has not been generally accepted. However, a number of studies have been made in which the effects of repeated loads on deformation and elastic modulus of asphalt mixtures have been established. These studies, being limited in scope, can only serve as guidelines to further research.

Effects on Strain or Deformation

In investigating the effects of repeated loads on the strain or deformation of asphalt mixtures, both flexure and compression tests have been utilized.

Compression Tests. Wood and Goetz have investigated the effects of repeated loads on specimens of sheet-asphalt mixtures tested in unconfined and confined compression (Refs 79 and 80). Figure 15a shows the relationship between the number of load repetitions (log scale) and the permanent deformation for three stress levels tested in unconfined compression. The curves generally start out as a straight line and at some point, depending upon the applied stress, deviate sharply from it. This point, which Wood and Goetz termed the failure point, was said to correspond to a point of excessive shear deformation. They suggested that with load repetitions the asphalt film between particles is reduced in dimension until some critical thickness is reached, at which point, in order to sustain the load, an adjustment in the specimen takes place by reorientation of the aggregate particles themselves. Figure 15b shows the variations in elastic and permanent deformation with repetitive loading. A minimum value of permanent deformation per cycle (approximately 0.002 inch) is obtained at a point corresponding to failure. In the same figure elastic deformation is shown to be constant (about 0.016 inch) throughout the duration of the test. The same general trends were obtained for specimens subjected to confining pressures.

In a later work, however, Goetz et al (Ref 14) considered the physical appearance of the specimens and said that a plot of the logarithm of cumulative permanent deformation versus the logarithm of the number of load

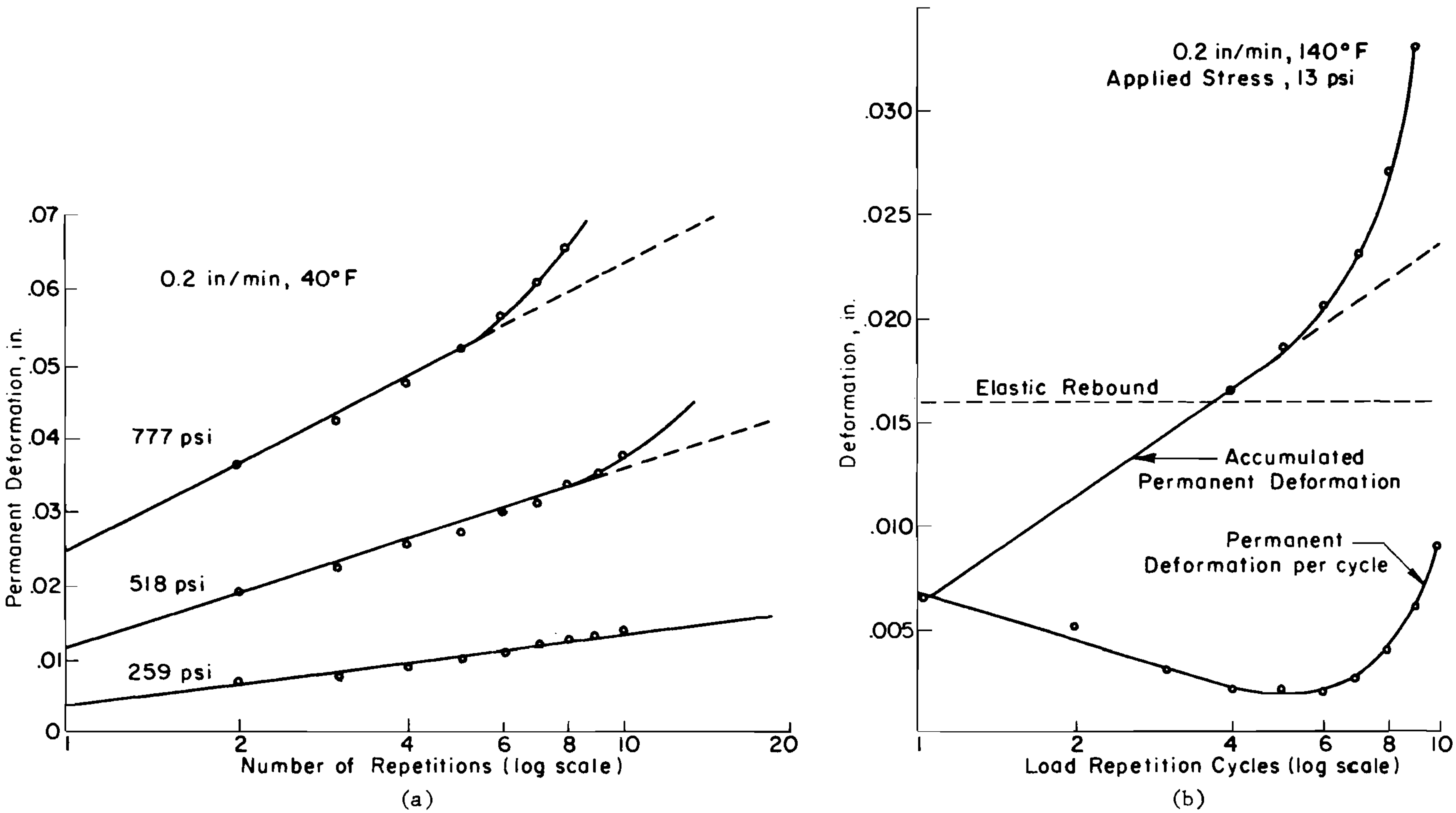


Fig 15. Relationship between elastic and permanent deformation with repetitive loading (Ref 79).

repetitions is the best method of interpreting the data and suggested that failure occured at the point of curvature. They suggested an equation of the form

$$Y = Kx^{\frac{1}{n}}$$

or

$$\log Y = K' + \frac{1}{n} \log x \qquad (2.11)$$

where

Y = cumulative permanent deformation,
x = number of load applications, and
K, K', and n are constants.

The slope of the straight line on a log-log plot of the above equation $\frac{1}{n}$ was termed an index of plasticity. It was finally concluded that such a test might provide valuable information concerning the plastic nature of the mixture and, in addition, give a measure of its endurance limit.

Monismith and Secor (Ref 50) in investigating the thixotropic nature of asphalt paving mixtures with reference to behavior in repeated axial compressive loading obtained results similar to those obtained earlier by Wood and Goetz (Refs 79 and 80). Their data indicated that the magnitude of strain and the point of excessive increase in strain depend on the stress level. They also indicated that a slightly higher strain is obtained with a lower frequency of test.

From a compressive load test on cold mixes and at stress levels lower than those used by Monismith and Secor, Howeedy and Herrin (Ref 24) have obtained results in which the variations of residual (permanent) strain with number of stress applications were similar to those obtained by Monismith and Secor. They observed three distinct zones in the residual strain versus number of load application plots: (1) a straight line, (2) a slight curvature, and (3) an excessive curvature where the residual strain increased rapidly with little increase in the number of load applications.

In a recent study, Morris and Haas (Ref 54) reported that for specimens subjected to axial tensile and compressive strains under confining pressure, there is a "conditioning phase" in which a straight line increase of permanent strain was obtained with increasing load repetitions. They went further to establish predictive equations for the log slope of the straight line portions in terms of the axial strain, confining pressure, and testing temperature. However, they did not indicate what happens in the failure zone nor did they, like others, indicate what proportion of the fatigue life relationship can be represented by the straight line.

Flexure Tests. Papazian and Baker presented results illustrating the influence of load repetitions on the deflection of the center of beam specimens resting on a transverse simply supported steel leaf (Ref 56). These results indicated that the deflection is independent of the number of load applications up to a certain point, depending on the stiffness and the supporting span of the steel leaf, after which the deflection increases rapidly for a few additional load applications.

Jimenez and Gallaway performed repetitive loading tests on asphaltic concrete diaphragms (Refs 29 and 30). The relationship of deflection to the log of number of load applications was similar to that obtained by Wood et al (Refs 79 and 80) from compression tests. The point at which the plot deviates from the initial straight line was also taken as the failure point. To establish the straight portion, the load-disc deflections recorded within the first 1000 repetitions were disregarded since it was felt that these were subject to variabilities of initial disc location and seating and also adjustment of the specimen to load.

In his investigation of the fatigue of a bituminous mixture under compound loading, McElvaney plotted the deflection versus cycle ratio curves for the rotating cantilever beam specimens tested under repeated flexure (Ref 40). Figure 16 shows typical curves in which there is an initial curvature before the straight portion and the final curvature are obtained. Figure 16 also indicates the effect of stress level on the position of the curves. The general shape of the curves is similar to that previously obtained for maximum dynamic deflection by Deacon from beam tests (Ref 6).

Effects on Modulus of Elasticity

Many forms of modulus of elasticity of an asphalt mixture have been

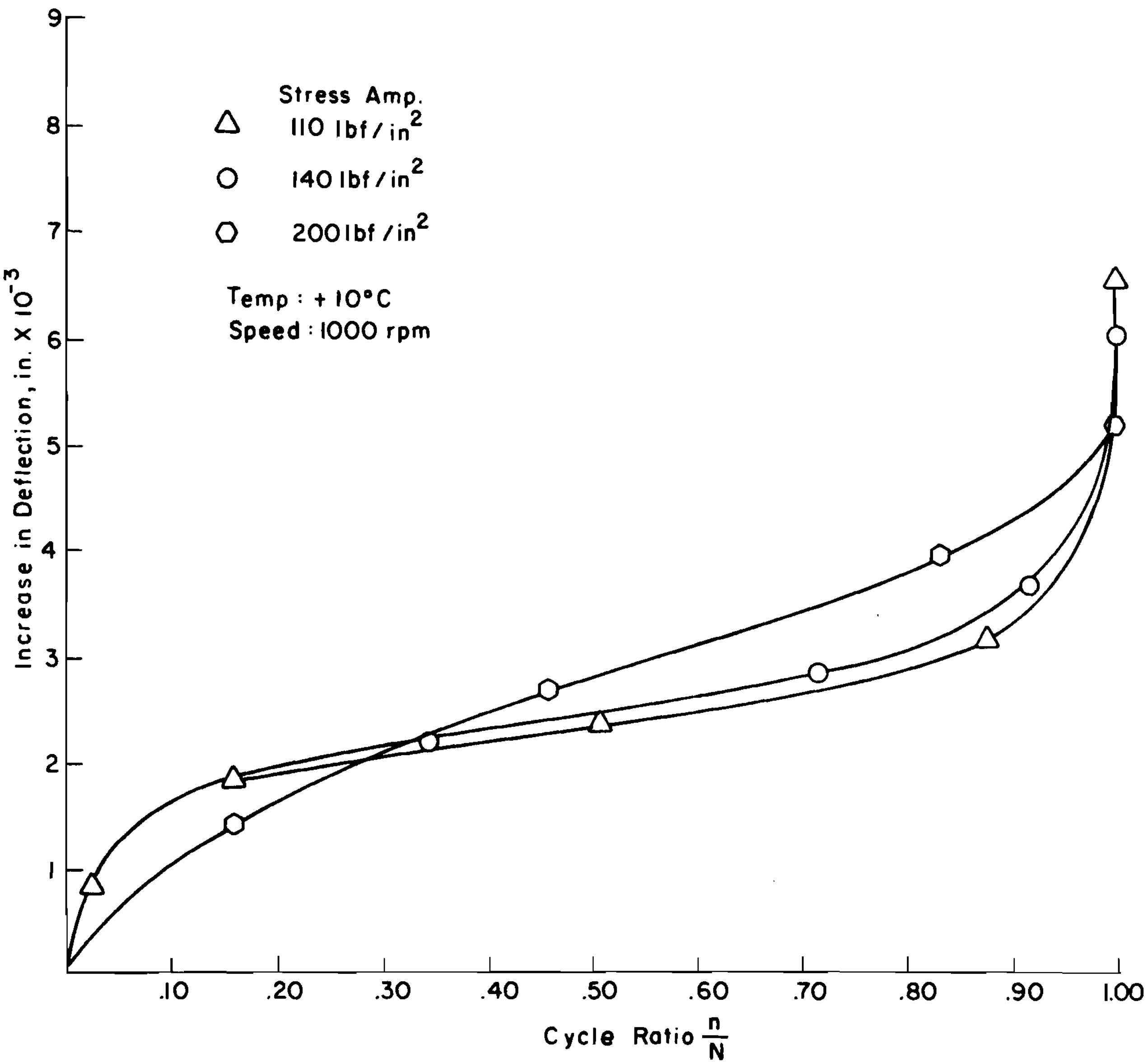


Fig 16. Increase in deflection versus cycle ratio (Ref 40).

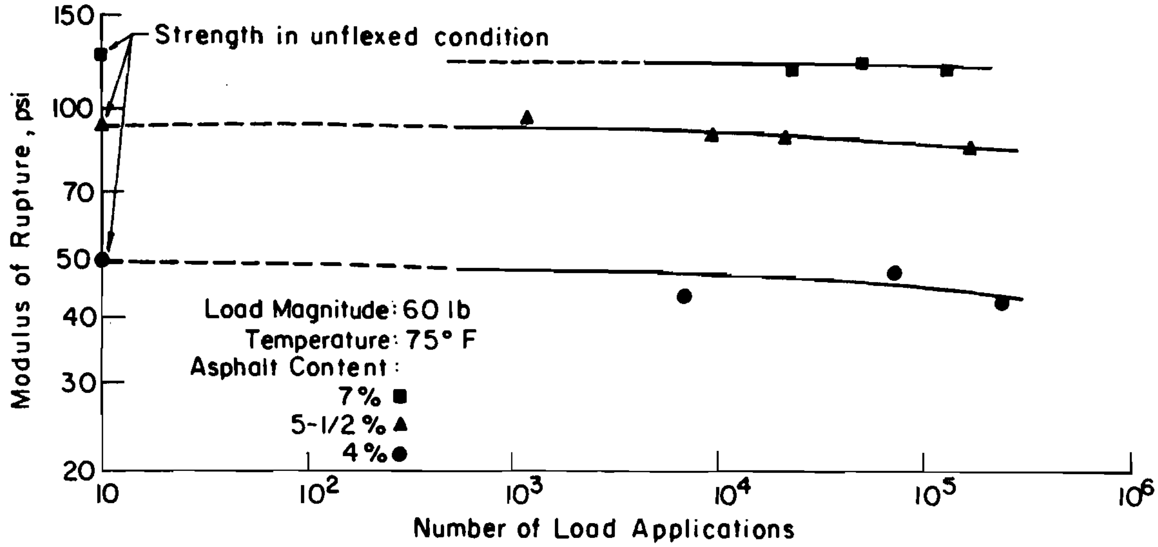
defined and estimated, including the following: (a) modulus of rupture, which is generally associated with beam or flexure tests and is a measure of the flexural strength of the specimen; it differs from any other form of modulus in that its estimation is based on a destructive test, thereby requiring many specimens to plot the modulus versus load repetitions curve; (b) stiffness modulus, which is simply the ratio of stress to strain, and the strain could be based on elastic deformation or total deformation; (c) resilient modulus, which is generally associated with triaxial (or biaxial) tests and is defined as the ratio of deviator stress to the recovered strain. Another form of modulus, of course, is the complex modulus, which is based on dynamic vibratory tests.

The stiffness or resilient modulus of asphalt mixtures can be estimated from non-destructive tests so that the entire modulus versus load repetitions curve can be obtained from a single specimen.

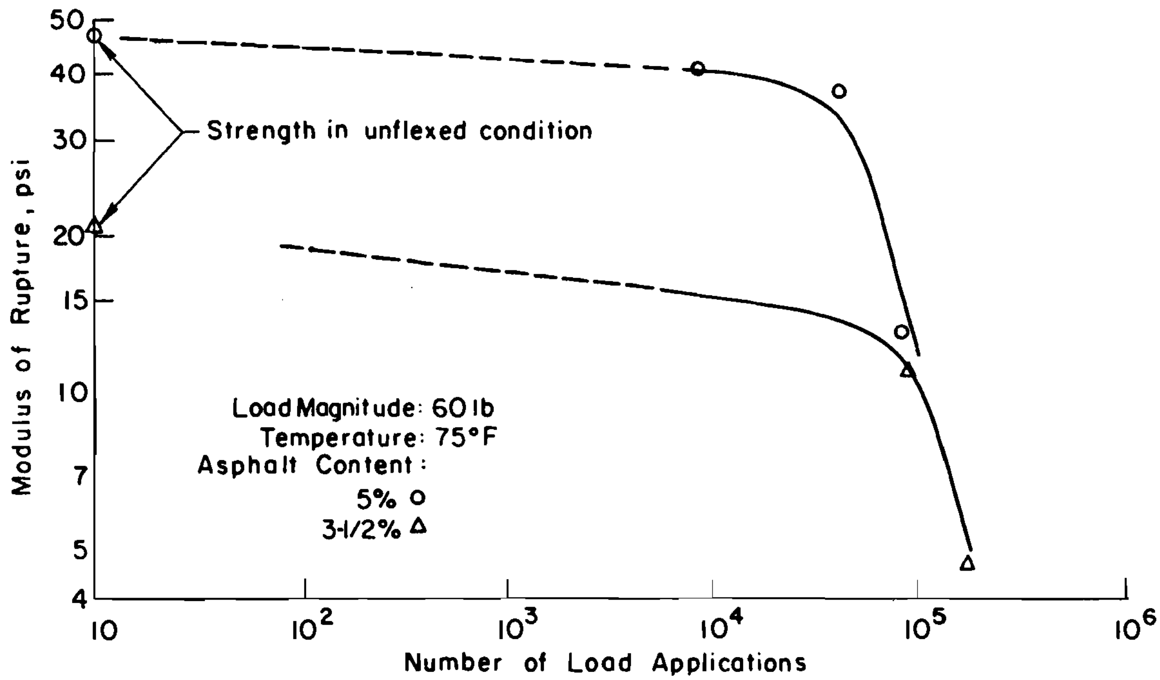
Both flexure and compression tests have been utilized in finding the effects of repeated loads on modulus of asphalt mixtures.

Flexure Tests. The results of various studies (Refs 6, 12, 22, 32, 42, and 49) have indicated that the modulus decreases with increasing load applications in a manner which depends on load, mixture, and environmental variables.

Monismith investigated the effects of repeated loads on the modulus of rupture of asphalt paving mixtures by subjecting beam specimens to various numbers of load applications and then statically loading them to failure (Ref 42). The specimens were rested on a spring base which was constructed to simulate the base-subgrade combination in an actual pavement structure. Tests were performed in a constant temperature room to eliminate the effect of temperature variation. Monismith's results indicated that for the dense graded mixtures and lower stress levels there is very slight reduction in strength with load applications whereas there is a marked reduction in strength for the open graded mixtures and higher stress levels. Also, the higher the asphalt content and the lower the testing temperature, the higher the modulus of rupture of the dense graded mixtures. Figure 17 illustrates the effects of asphalt content and gradation on reduction in stiffness with load applications.



(a) Dense gradation.



(b) Open gradation.

Fig 17. Effects of asphalt content and gradation on the behavior of asphalt mixtures subjected to repeated-load applications (Ref 42).

In investigating the effects of the rheological properties of asphalt on the strength characteristics of asphalt concrete, Hewitt and Slate also determined the influence of repetitive loading on flexural strength by determining the modulus of rupture after 1,000 and 100,000 load repetitions (Ref 22). They presented figures which indicated that the effects of repeated loads on the modulus of rupture are influenced by the type of asphalt used. They concluded that repeated loading produces a marked decrease in flexural strength of asphalt concrete and that this reduction may be attributed in part to a change in the physical properties of the asphalt concrete (deflection and density). A comparison of the results obtained by Hewitt and Slate with those previously obtained by Monismith for open graded mixtures shows that the decrease in modulus in the range from 10,000 to 100,000 load applications is more gradual for the results obtained by Hewitt and Slate than those obtained by Monismith. This difference is probably due to the differences in mixtures and loading conditions.

Like the modulus of rupture, repeated loading has been found to have a marked effect on the stiffness modulus of asphalt mixtures. Deacon (Ref 6) has indicated that for any asphalt specimen subjected to a simple loading test the stiffness modulus (based on deflection) initially decreased quite rapidly with repeated loads, followed by a prolonged, more gradual, and approximately linear decrease and finally a more rapid decrease towards fracture condition. This trend is illustrated in Fig 18, where the stiffness was plotted against the number of load applications. By plotting the stiffness-versus-cycle ratio, Deacon was able to compare the manner in which the stiffness moduli change as a function of the accumulation of load repetitions for different stress levels. This is illustrated in Fig 19, from which he observed that the slopes of the linear portions of the curves increase with increasing applied stress.

The effect of stress applications on the stiffness of asphalt mixtures representing the Morro Bay pavement has been investigated under the controlled-strain test by Monismith et al (Ref 49). They presented results which indicated a more gradual reduction in stiffness in the failure region than those previously obtained by Monismith for the modulus of rupture (Ref 42). This might, however, be explained by the slower rate of crack propagation in strain-controlled tests.

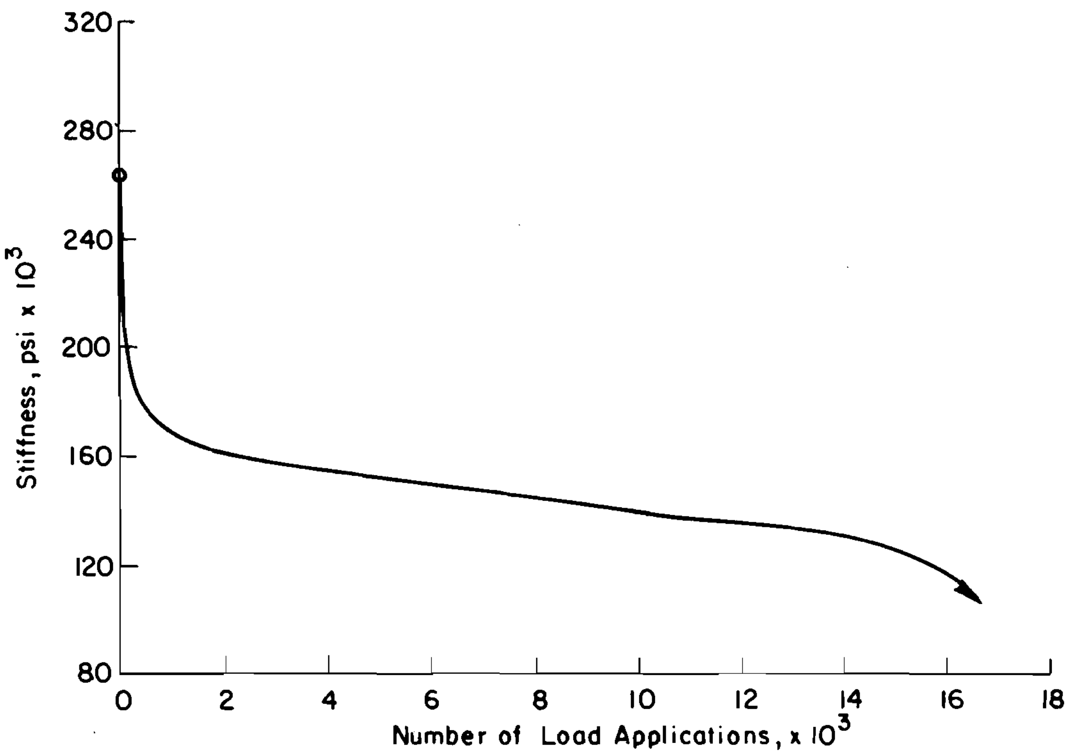


Fig 18. Reduction in stiffness with increasing load applications (Ref 6).

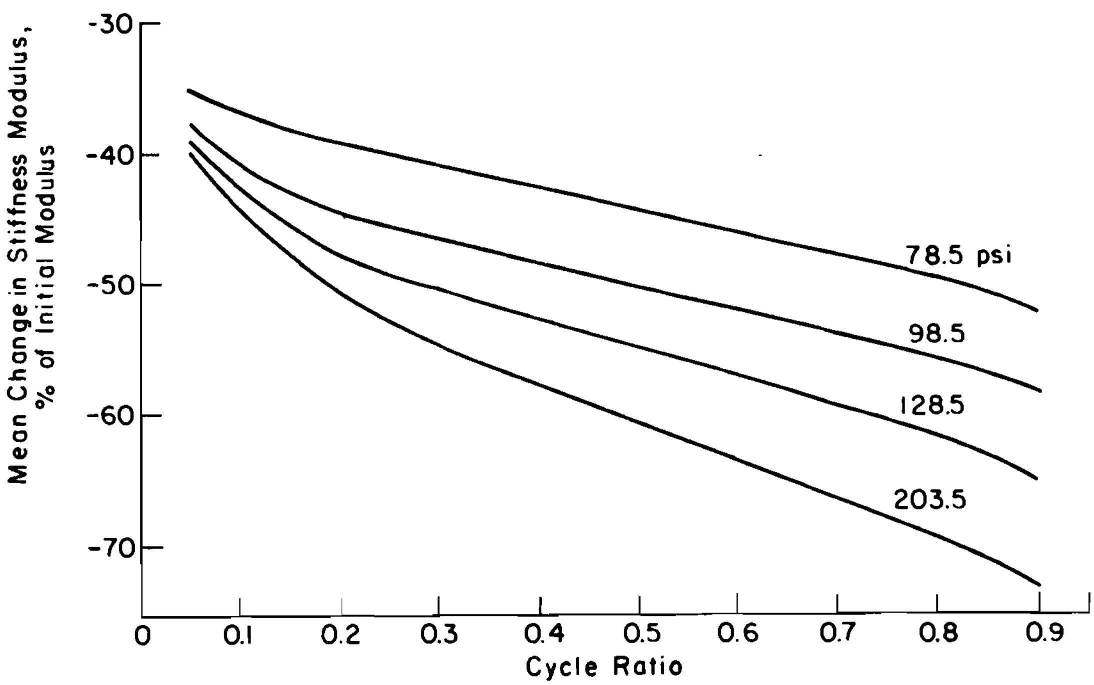


Fig 19. Alteration of mean stiffness modulus with cycle ratio, simple loading (Ref 6).

Kallas and Puzinauskas have also found that the deflection-based stiffness modulus of a Colorado asphalt concrete surface mix tested at 70°F decreased during controlled-stress flexural fatigue tests (Ref 32). They found that the modulus decreased at a faster rate with increasing stress level, a result similar to that earlier obtained by Deacon (Ref 6).

In research completed in South Africa, Freeme and Marais utilized trapezoidal specimens on which a half-sine-wave impulse was applied for both gap-graded and continuously graded bituminous mixtures (Ref 12). Figure 20a shows a typical representation of reduction in peak stiffness with number of load applications for the gap-graded specimens tested under the controlled-strain test at 20°C. The reduction in peak stiffness was divided into three zones: initial rapid reduction in stiffness, crack initiation, and crack propagation zones. In Fig 20b the rates of decrease in stiffness with the logarithm of number of repetitions for both the crack initiation zone R_i and the crack propagation zone R_p are shown. These quantities are related by the general formula:

$$S = R \log N + S_1 \tag{2.12}$$

where

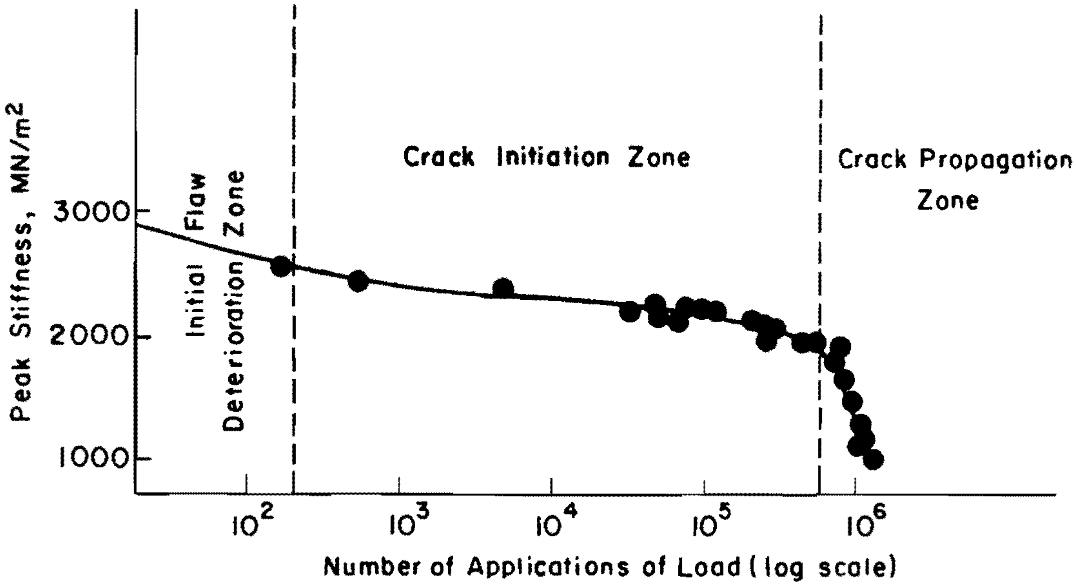
S = stiffness,

N = number of load repetitions, and

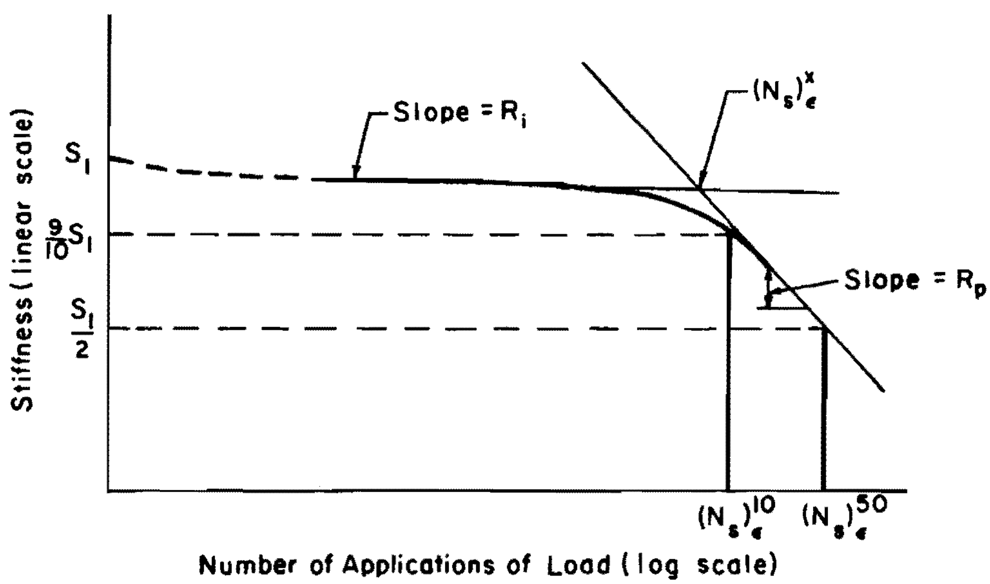
S_1 = initial stiffness or stiffness for the first application.

The dependence of R_i and R_p on temperature is illustrated in Fig 21. It is observed that for both the gap-graded and the asphalt concrete mixtures, R_p decreases with increasing temperature. On the other hand, there seems to be an optimum temperature (20°C) for a maximum value of R_i for both asphalt mixtures.

Based on the prediction of crack initiation and crack propagation life in an experimental pavement using the National Institute of Road Research Fatigue Prediction Program, Freeme and Marais (Ref 12) found that if a particular pavement has a short fatigue life, the stiffness of the surface will deteriorate very rapidly in the crack propagation zone and that if the crack



(a) Division into zones



(b) Definitions of service life

Fig 20. Variation of specimen stiffness with number of load applications (Ref 12).

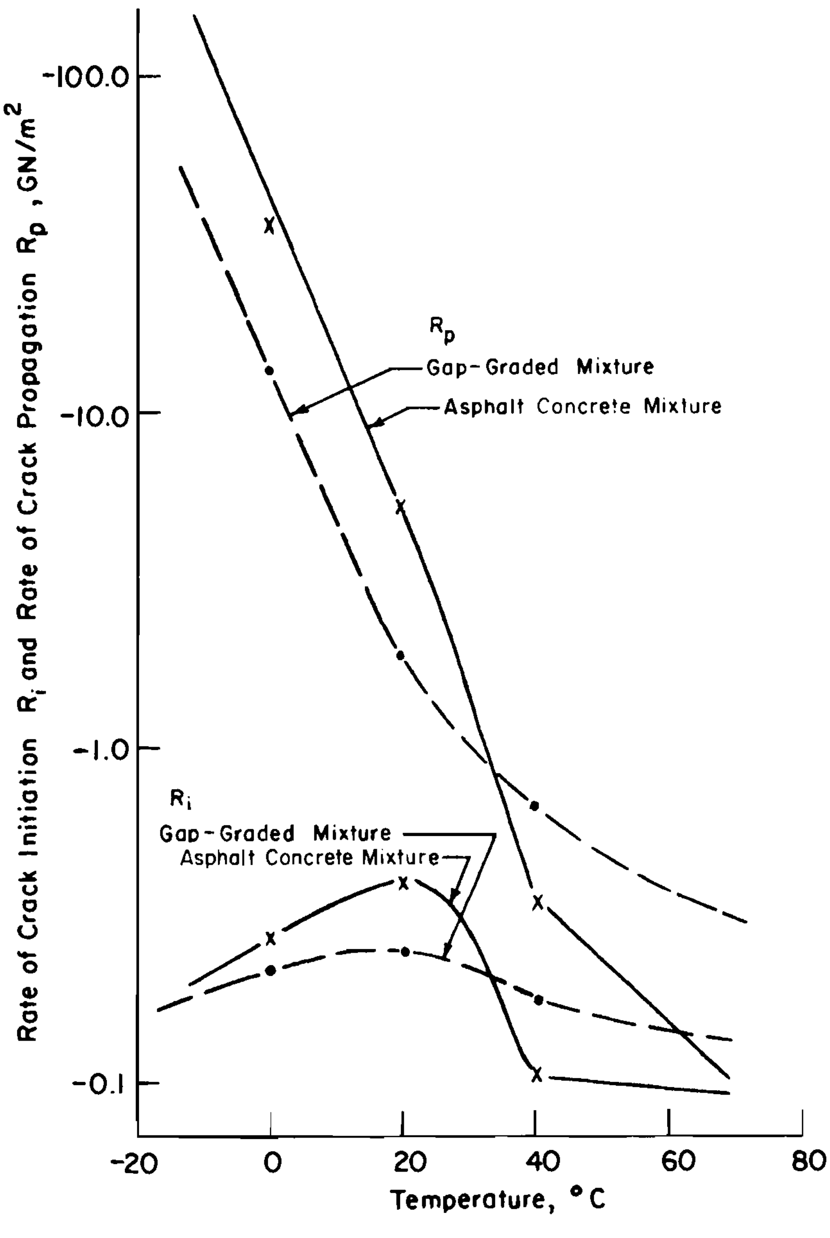


Fig 21. Variation of R_i and R_p with temperature (Ref 12).

initiation period is long, the surface stiffness will deteriorate slowly.

Tension/Compression Tests. Raithby and Sterling have investigated the fatigue resistance of hot-rolled asphalt under reversed axial loading (Ref 64). Their normalized plot of compressive and tensile stiffness moduli is shown in Fig 22. There is an initial more rapid change of tensile modulus after which the rate of change of stiffness is much the same in both the tensile and compressive parts until the last 20 percent of the life, when the tensile stiffness decreased more rapidly than the compressive. They observed that this stage corresponds to the point at which visible cracks appeared in the surface of the test specimen. The general shape of the tensile stiffness curve is similar to that obtained by Deacon from flexure tests (illustrated in Fig 18).

The work of Howeedy and Herrin (Ref 24) on cold mixes under repeated compressive loads showed that the value of the modulus of resilience appeared to be constant in the vicinity of 1 percent residual strain. At about 1.5 percent residual strain there is a sudden decrease in the resilient modulus. This trend is illustrated in Fig 23.

Effects on Poisson's Ratio

Various factors have been found to affect the values of Poisson's ratio for asphalt concrete mixtures under static test. Hadley et al (Ref 18) have found that compaction temperature and asphalt content are the only two factors while aggregate type interacting separately with asphalt content and with aggregate gradation are factors which affect the values of Poisson's ratio to a significant level. However, the changes in Poisson's ratio as a result of repeated loads on asphalt concrete mixtures have either been not investigated or not reported. Since this knowledge might be important in arriving at a design value which could be utilized in the elastic layer theory, any studies in this direction would be valuable.

Effects on Tensile Strength

The effects of repeated loads on the tensile strength of asphalt concrete mixtures have not been reported. This might be due to the fact that ultimate strength is generally believed not to be a fatigue life determinant since specimens fail at stresses quite below the ultimate tensile strength

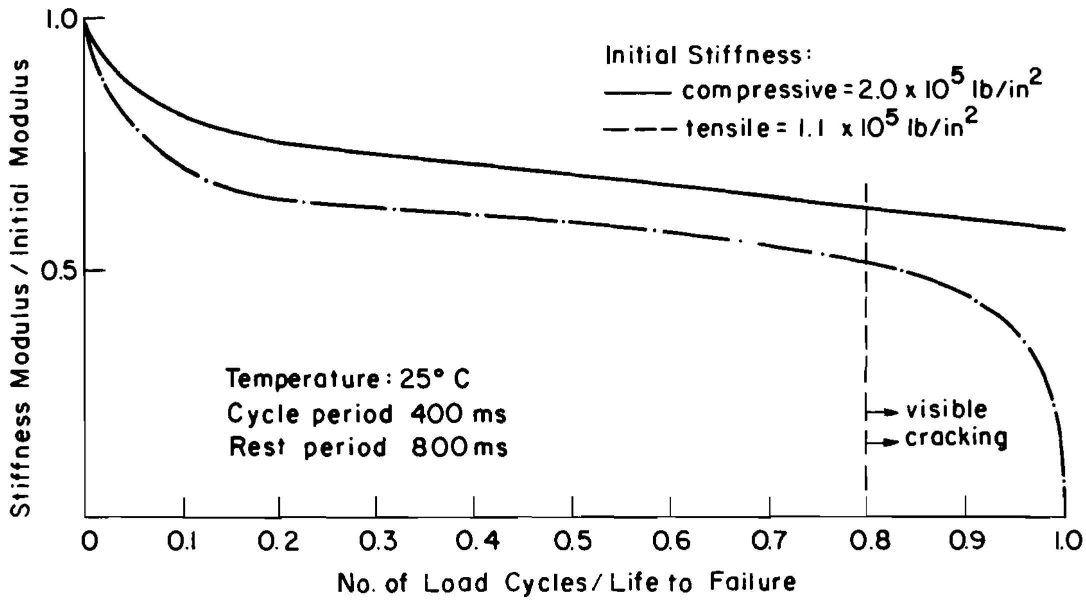


Fig 22. Effect of repeated loading on compressive and tensile stiffness moduli (Ref 64).

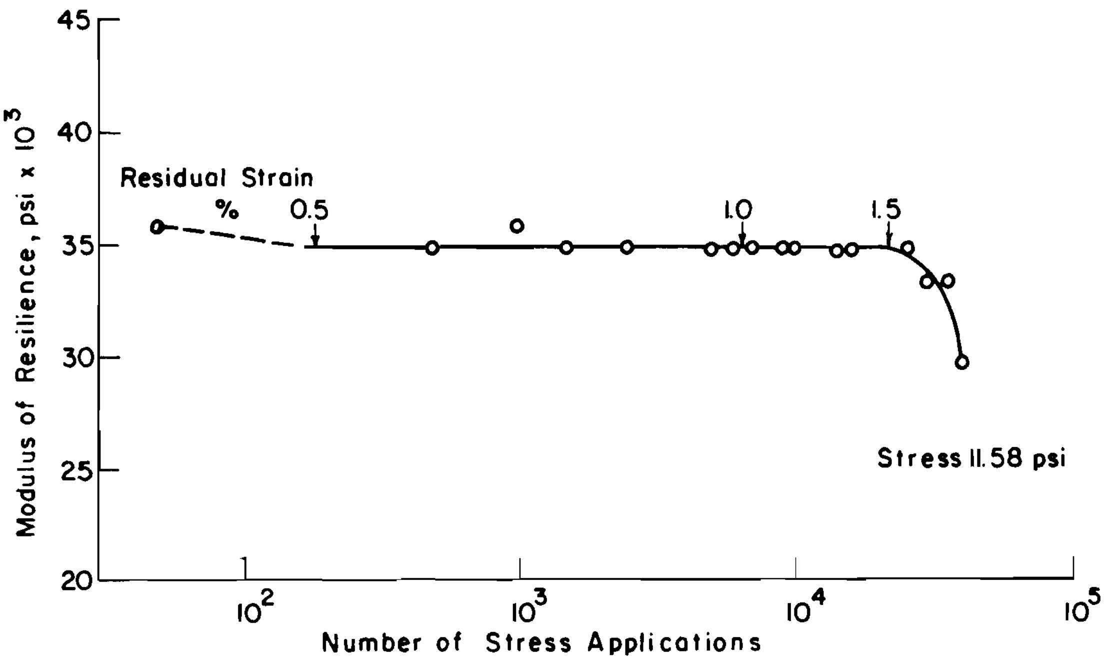


Fig 23. Variation of modulus of resilience with number of stress applications (Ref 24).

if repeated long enough. Also, like modulus of rupture tests, tensile strength tests are destructive, requiring many specimens to define a tensile strength versus load repetitions curve. However, it seems that knowledge of the variation of tensile strength with repeated-load applications would be useful in predicting initiation of cracks, and any such laboratory findings could be correlated with actual field pavement behavior and thus constitute a potential help in pavement overlay design and maintenance.

Asphalt Cement

A brief review of the effects of repeated loads on the properties of asphalt cement is presented since this may, to some extent, explain the behavior of asphalt mixtures subjected to repeated loads.

Pell (Ref 57) studied the effects of repeated bending stresses and torsional strains on bitumen alone and found that a linear relationship exists for the logarithm of number of load repetitions and the logarithm of stress or strain. He observed that at low stresses, the measured fatigue life includes a considerable length of crack propagation time. He found that unlike the sandsheet specimens which he also tested, bitumen alone showed beneficial effects of rest periods, particularly at high temperatures.

In addition, Lou has investigated the effects of repeated loads on the viscosity of paving asphalts in thin films (Ref 37). He concluded that whereas the viscosity of the specimens of asphalt studied increased about 20 to 40 percent during the first 72 hours after preparation if allowed to stand undisturbed, the viscosity of comparable specimens subjected to repeated loading increased only about 8 to 15 percent. Both magnitude and frequency of load application affect the rate of change in viscosity, as illustrated in Fig 24.

Summary and Comments

The effect of repeated loads on the tensile and elastic properties of asphalt mixtures is complex, depending on load, mixture, and environmental variables.

In general, permanent deformations increase gradually, approximating a straight line, until a sharp increase is obtained at the failure zone. The elastic deformation, on the other hand, is almost invariant with load repetitions.

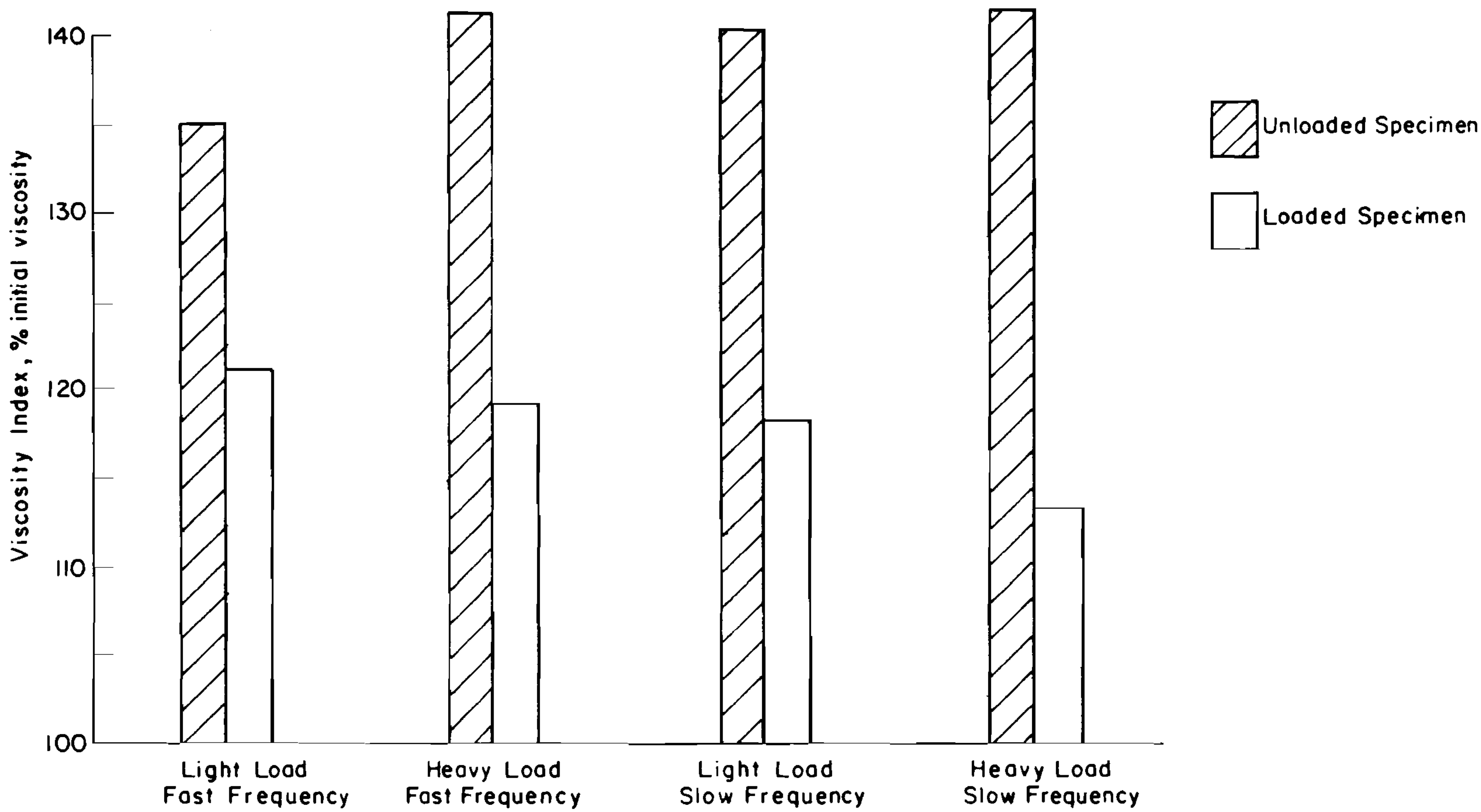


Fig 24. Influence of repeated load applications on asphalt "T" after 72 hours (Ref 37).

Similarly the modulus of elasticity generally decreases with load repetitions, with a sharp decrease in the failure zone. The initial shape of the curve may vary anywhere from a sharp decrease to a horizontal curve.

The type of test employed has no substantial effect on the general shapes of the curves. Finally the behavior of asphalt cement under repeated loads is very similar to that of an asphalt mixture.

It is observed that only the changes in the modulus of elasticity and strain (deformation) of stabilized pavement materials under repeated loads have been reported. The effects of repeated loads on Poisson's ratio and tensile strength of asphalt mixtures are not yet known. The methods of test have mainly been flexure and compression. It seems, however, that the results based on compression tests would not be realistic for pavements since fatigue cracking is tension failure and not compression. Test methods measuring tensile properties are therefore more desirable for fatigue prediction while compression tests are more suitable for measuring permanent deformations (rutting prediction). In addition, it is felt that the methods used are time-consuming and therefore difficult to use for day-to-day design and analysis. A simple test method such as the indirect tensile test will be more suitable.

CHAPTER 3. EXPERIMENTAL PROGRAM

The objectives of this study, as presented earlier, include the following: (1) to utilize the indirect tensile test method in finding the effects of mixture variables and temperature on the fatigue behavior of asphalt mixtures, (2) to establish fatigue life prediction equations in terms of load and mixture properties for the mixtures studied, and (3) to estimate the effect of repeated loads on the tensile and elastic (or resilient) characteristics of asphalt mixtures.

Accordingly, an experimental program dealing with materials, specimens, test equipment, experimental design, testing procedure, and analytical approach was carried out as described in this chapter.

MATERIALS

Aggregate

Two types of aggregate with contrasting properties were used: an angular and relatively porous crushed limestone supplied by the Texas Crushed Stone Co. from a location five miles south of Georgetown, Texas; and a rounded, smooth, and relatively nonporous gravel obtained from a location near the Seguin River in Texas. The gradation for both aggregates is shown in Fig 25 and is the same as the medium gradation used by Moore and Kennedy in an earlier project (Refs 52 and 53). The gradation conformed with the Texas Highway Department Standard Specifications for hot mix asphaltic concrete pavement Class A, Type B (fine graded base or leveling-up course) and Type C (coarse graded surface course) (Ref 72) except for the portion passing the No. 200 sieve, which is a little on the high side of both specifications, and the cumulative total retained on sieve No. 10, which is lower than the specified lower range in specification Type B. Table 2 shows a comparison of the gradation used with the two specifications described above.

Asphalt

The asphalt was an AC-10 asphalt cement produced by the Cosden Refinery,

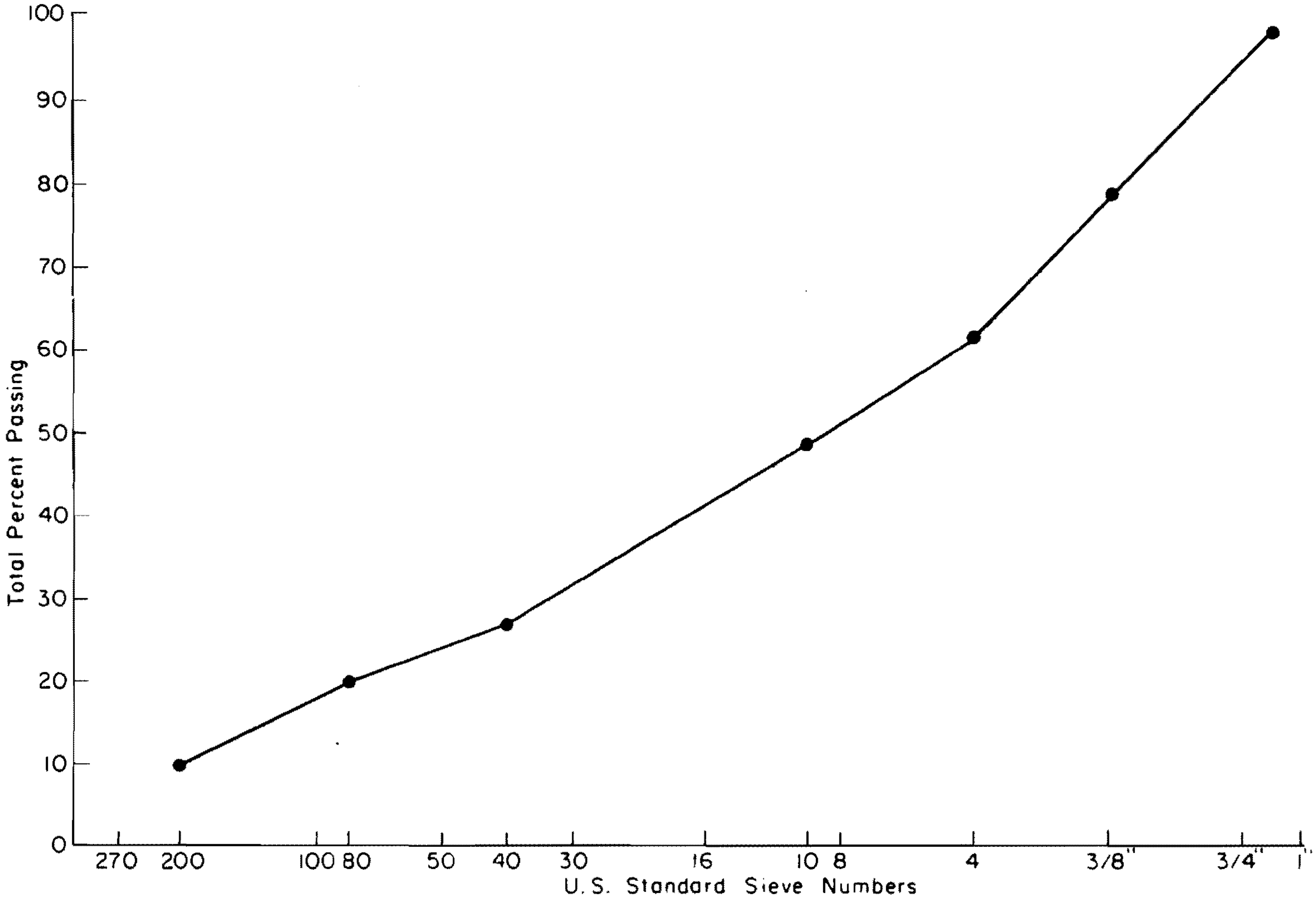


Fig 25. Aggregate grading curve.

TABLE 2. COMPARISON OF GRADATION WITH THD SPECIFICATION FOR HOT MIX ASPHALT CONCRETE (HMAC) PAVEMENT, CLASS A, TYPE B (FINE GRADED BASE OR LEVELING-UP COURSE) AND TYPE C (COARSE GRADED SURFACE COURSE).

	Percent Aggregate, by Weight		
Sieve Size	THD Class A, Type B	THD Class A, Type C	Gradation Used
+ 7/8"	95 - 100	100	
- 7/8" + 5/8"	20 - 50		20
- 5/8" + 3/8"		15 - 40	
- 3/8" + #4	10 - 40	10 - 35	18
- #4 + #10	5 - 25	10 - 30	13
Total + #10	55 - 70	50 - 70	51
- #10 + #40	0 - 30	0 - 30	22
- #40 + #80	4 - 20	4 - 25	7
- #80 + #200	3 - 20	3 - 25	10
- #200	0 - 6	0 - 6	10
Range of asphalt content, % by wt. of mixture	3.5 - 7	3.5 - 7	4 - 8

Big Spring, Texas. The properties of the asphalt cement as determined by the Texas Highway Department are summarized in Table 3. Asphalt contents varied from 4 to 8 percent by weight of total mixture.

PREPARATION OF SPECIMENS

The method adopted in manufacturing the specimens was essentially the same as that used in earlier research projects conducted at the Center for Highway Research (Refs 5, 16, 18, and 52). Aggregates were batched by dry weight to meet the specified gradation. Both the aggregate and a sufficient quantity of asphalt cement were heated to 275°F ± 5°F, after which the mixtures of aggregate and asphalt cement were mixed for about three minutes in an automatic, 12-quart-capacity Hobart mixer.

The mixtures were then placed in preheated ovens and brought to the compaction temperature of 250°F ± 5°F, after which they were compacted using the Texas gyratory-shear compactor, following the procedure specified in test method Tex-206-F, Part II (Ref 38). After compaction, the specimens to be tested at 75°F were cured for two days at room temperature (75°F ± 2°F) before testing. For tests conducted at 50 and 100°F, specimens were cured at room temperature for 24 hours and then transferred to temperature controlled rooms and kept at 50°F ± 2°F or 100°F ± 2°F for another 24 hours before testing.

SPECIMEN CHARACTERISTICS

All specimens were nominally 4 inches in diameter by 2 inches high. The bulk specific gravity and the bulk density of each specimen were estimated before it was subjected to load application. Using the bulk specific gravity of the aggregates, the apparent specific gravity of asphalt cement, and the bulk specific gravity of the specimens, the percent air voids at the asphalt content utilized were determined. Table 4 summarizes the mean bulk density, percent air void, and coefficients of variation of the specimens. Bulk density and air void content for individual specimens subjected to static tests are contained in Appendix B, and similar values of specimens subjected to repeated load tests are contained in Appendix C.

Figure 26 illustrates the relationship between average bulk density and asphalt content for limestone and gravel mixtures. The corresponding scatter diagrams are contained in Appendix B. As Fig 26 indicates, the limestone

TABLE 3. PROPERTIES OF ASPHALT CEMENT

Item	Number
Water, %	Nil
Viscosity at 275°F, stokes	2.45
Viscosity at 140°F, stokes	940
Solubility in CCl_4, %	—
Flash point C.O.C., °F	585
Ductility at 77°F, 5 cm/min, cm	—
Penetration at 77°F, 100 g, 5 sec	88
Tests on residues from thin film oven test:	
Viscosity at 140°F, stokes	2052
Ductility at 77°F, 5 cm/min, cm	141+
Residual penetration at 77°F	52
Original specific gravity at 77°F	1.031

TABLE 4. MEAN VALUES AND COEFFICIENTS OF VARIATION OF BULK DENSITY AND AIR VOID CONTENT FOR LABORATORY SPECIMENS

Aggregate	Asphalt Content, %	Type of Test	Number of Specimens	Bulk Density Mean, pcf	Bulk Density Std. Dev.	Bulk Density CV, %	Air Void Content Mean, %	Air Void Content Std. Dev.	Air Void Content CV, %
Limestone	4	Static	12	139.3	0.59	0.4	9.8	0.38	3.9
		Fatigue	15	139.6	0.45	0.3	9.6	0.29	3.0
		Total	27	139.4	0.53	0.4	9.6	0.34	3.5
	5	Static	12	141.8	0.77	0.5	6.8	0.51	7.5
		Fatigue	20	141.6	0.63	0.4	6.9	0.41	5.9
		Total	32	141.6	0.68	0.5	6.9	0.45	6.5
	5½	Static	3	143.2	0.55	0.4	5.2	0.36	7.0
		Fatigue	–	–	–	–	–	–	–
		Total	3	143.2	0.55	0.4	5.2	0.36	7.0
	6	Static	12	145.3	0.87	0.6	3.1	0.58	18.6
		Fatigue	45	145.5	0.81	0.6	3.0	0.54	18.1
		Total	57	145.4	0.82	0.6	3.0	0.55	18.3
	7	Static	19	146.2	0.42	0.3	1.1	0.28	25.5
		Fatigue	36	145.7	0.47	0.3	1.4	0.32	22.2
		Total	55	145.9	0.51	0.3	1.3	0.34	25.8
	8	Static	12	144.8	0.40	0.3	0.7	0.27	39.1
		Fatigue	14	144.6	0.46	0.3	0.9	0.31	36.0
		Total	26	144.7	0.44	0.3	0.8	0.30	38.5
Gravel	4	Static	12	136.8	0.59	0.4	10.1	0.39	3.9
		Fatigue	14	137.0	0.68	0.5	10.0	0.45	4.5
		Total	26	136.9	0.64	0.5	10.0	0.42	4.2
	5	Static	12	139.2	0.64	0.5	7.2	0.43	6.0
		Fatigue	21	139.6	1.08	0.8	6.9	0.72	10.4
		Total	33	139.5	0.96	0.7	7.0	0.64	9.1
	6	Static	12	142.1	0.89	0.6	4.0	0.60	15.0
		Fatigue	46	143.4	0.92	0.6	3.1	0.62	20.3
		Total	58	143.2	1.07	0.7	3.2	0.72	22.2
	6½	Static	5	144.2	0.58	0.4	1.9	0.39	20.9
		Fatigue	–	–	–	–	–	–	–
		Total	5	144.2	0.58	0.4	1.9	0.39	20.9
	7	Static	19	143.8	0.52	0.4	1.5	0.36	24.8
		Fatigue	40	143.9	0.42	0.3	1.4	0.29	20.7
		Total	59	143.9	0.45	0.3	1.4	0.31	21.8
	8	Static	12	143.4	0.27	0.2	0.4	0.19	47.5
		Fatigue	14	143.5	0.52	0.4	0.4	0.33	89.2
		Total	26	143.5	0.42	0.3	0.4	0.21	55.3

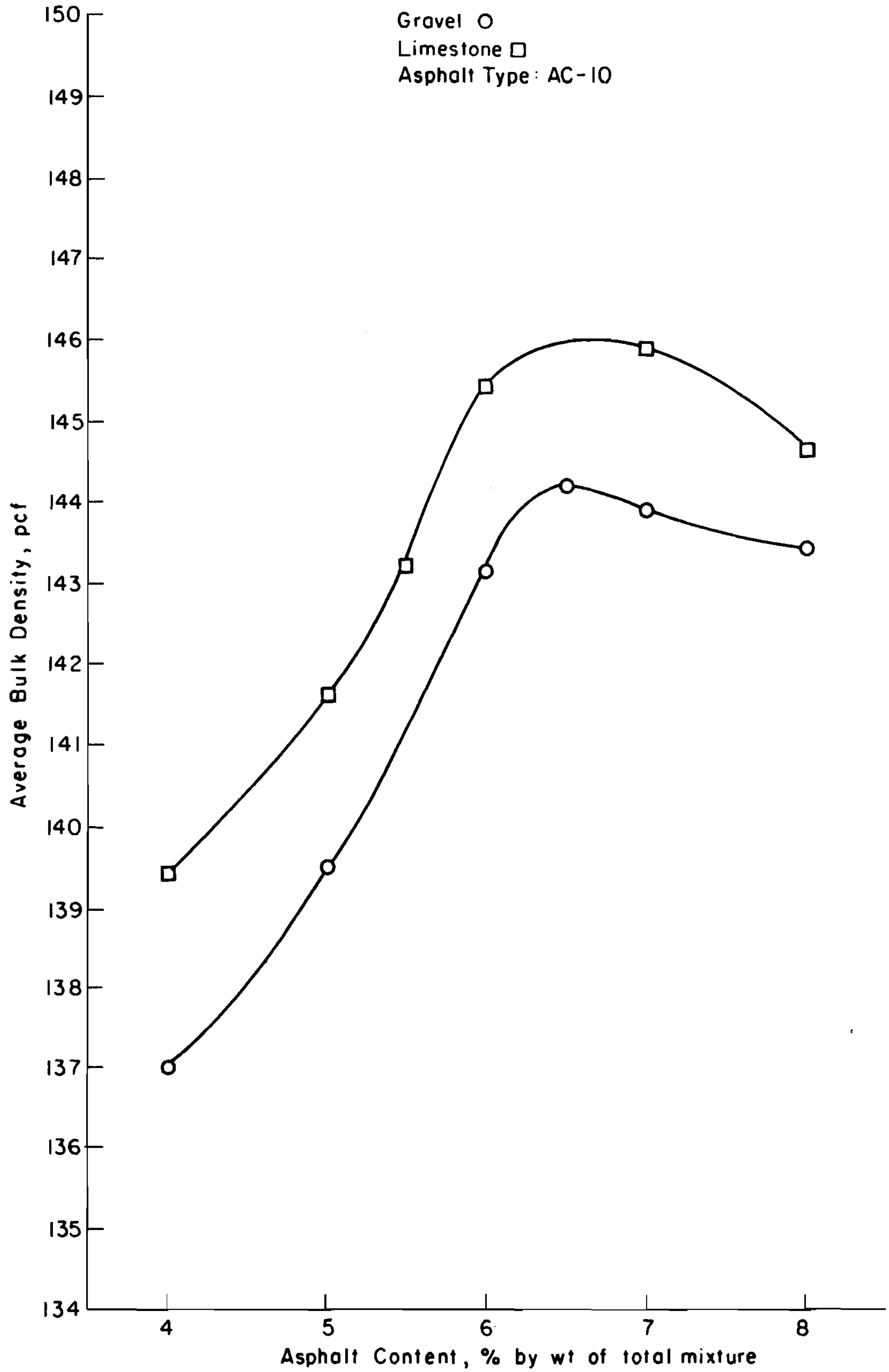


Fig 26. Relationship between average bulk density and asphalt content.

mixtures had a maximum density of 146 pcf at an optimum asphalt content of 6.7 percent, and the gravel mixtures had a maximum density of 144.2 pcf at an optimum asphalt content of 6.5 percent. The optimum asphalt content for the limestone mixtures was slightly greater than that for the gravel mixtures, possibly because of the angularity and rough surface texture of the limestone aggregates, which cause them to absorb a greater amount of asphalt than the rounded and relatively smooth gravel aggregates. The relationship between average air void content and asphalt content is illustrated in Fig 27.

TEST EQUIPMENT

The test equipment utilized for this study was basically the same as that previously used in a number of studies at the Center for Highway Research, the only difference being the recording system for the repeated load tests.

The equipment consisted of an adjustable loading frame, a loading head, and an MTS closed-loop electrohydraulic loading system (Fig 28). The loading head was obtained by modifying a commercially available die set with upper and lower platens constrained to remain parallel during testing (Fig 29). A curved stainless steel loading strip was attached to both the upper and lower platens. The dimensions and configuration of the curved loading strip together with specification details for the die set and loading strips are contained in Ref 1.

Estimation of the modulus of elasticity, Poisson's ratio, and tensile strains required the measurement of both vertical and horizontal deformations of the specimens. The vertical deformation was measured by a DC linear variable differential transducer (Schaevitz Engineering Type 1000 DC-LVDT). The permanent horizontal deformations were measured by using a special measuring device consisting of two cantilevered arms with strain gages attached (Fig 30). Movements of the arms at the point of contact with the specimen were calibrated with the output from the strain gages by using actuating screws and dial gages.

In the case of static tests, the load-horizontal and load-vertical deformations were recorded on a pair of X-Y plotters, Hewlett-Packard Model 7001 A. Three DC power supplies (Hewlett-Packard Model 801C) were utilized to power the load cell, the horizontal deformation device, and the LVDT.

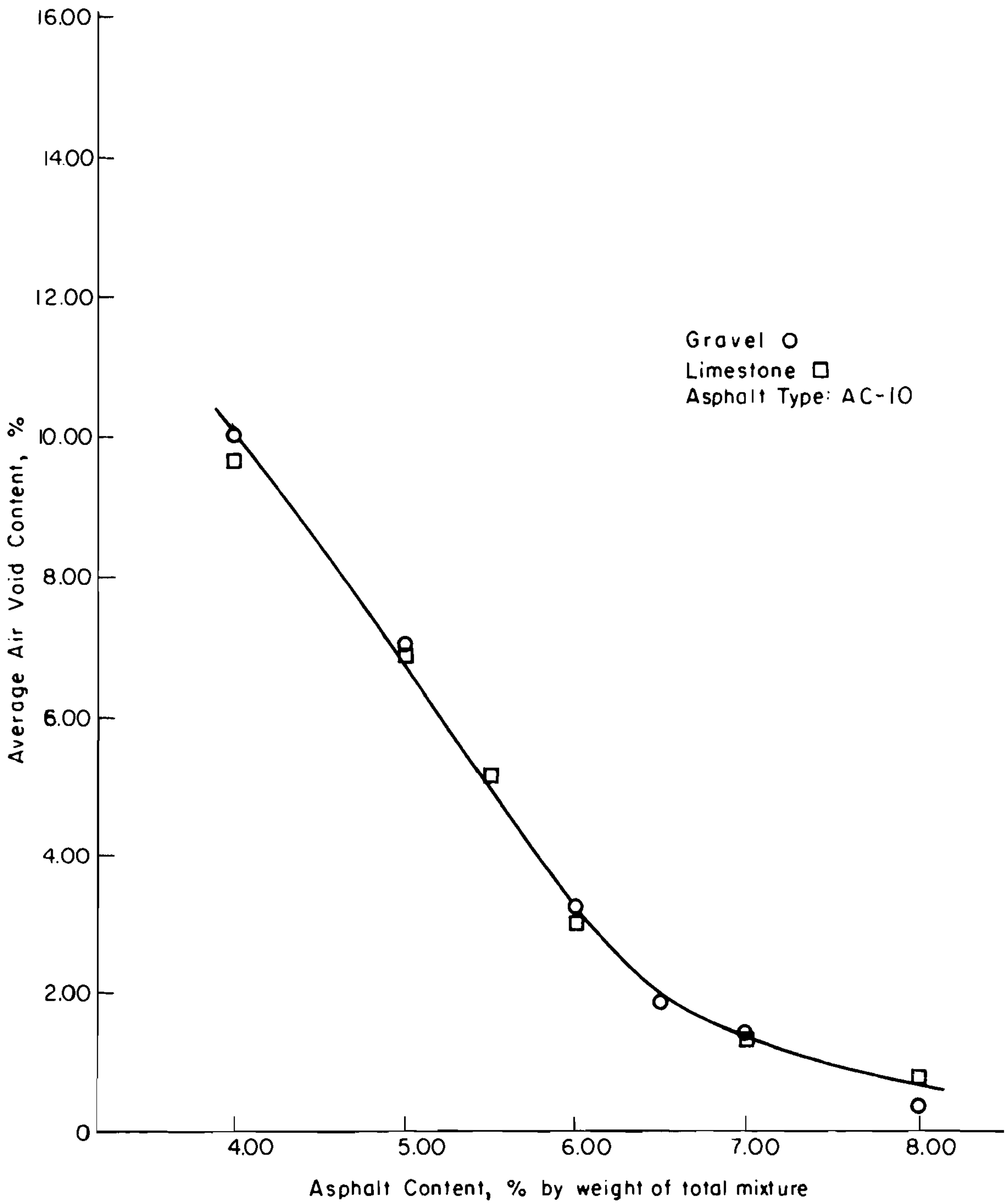


Fig 27. Relationship between asphalt content and average air void content.

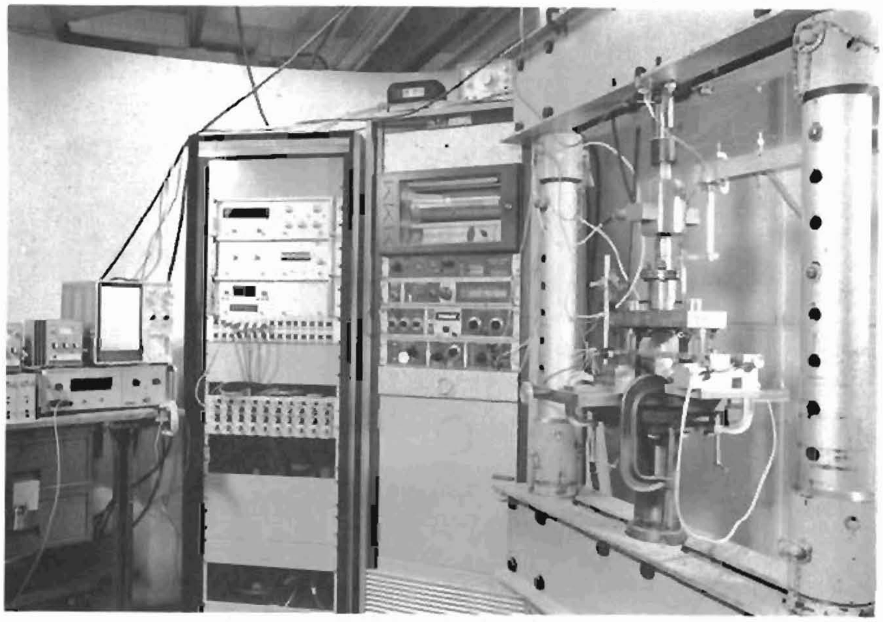


Fig 28. Basic testing equipment.

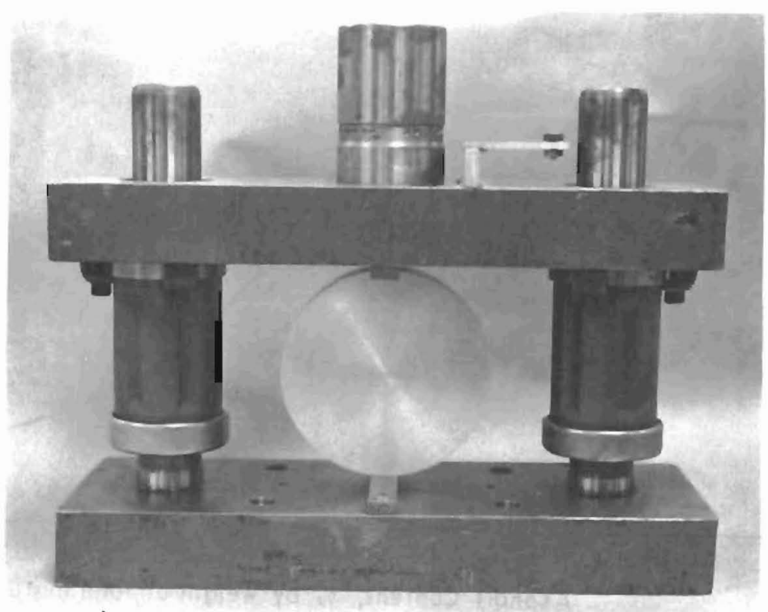
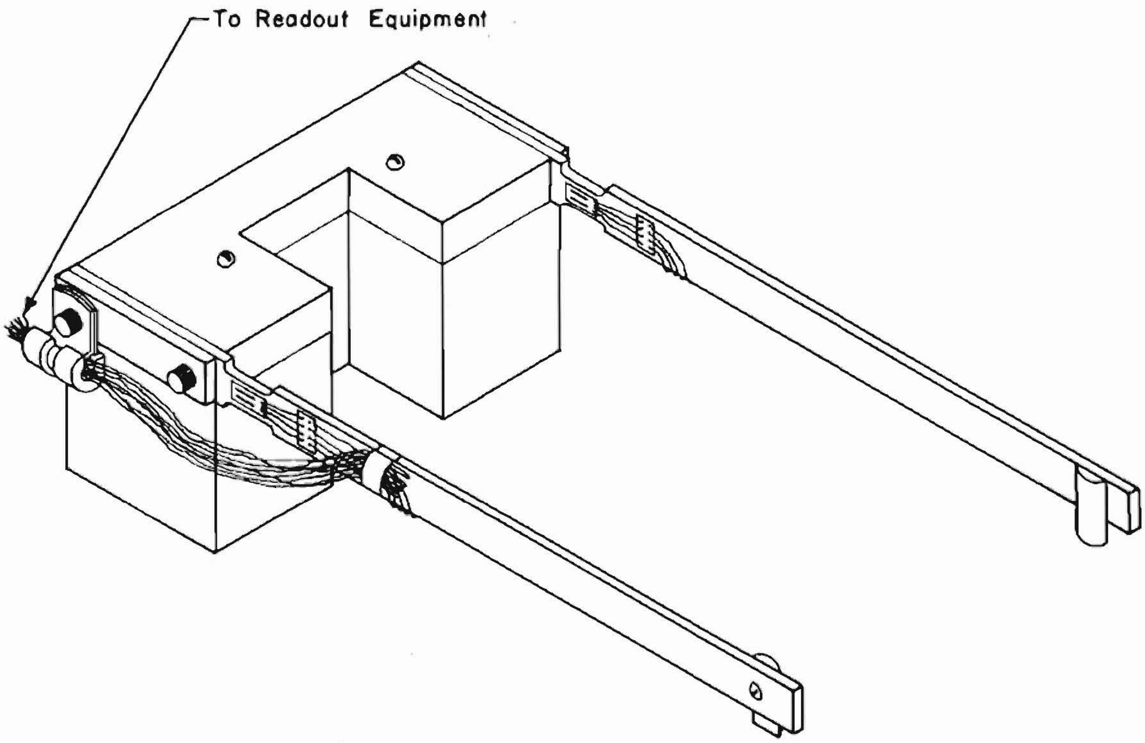


Fig 29. Loading head with rigid parallel platens.



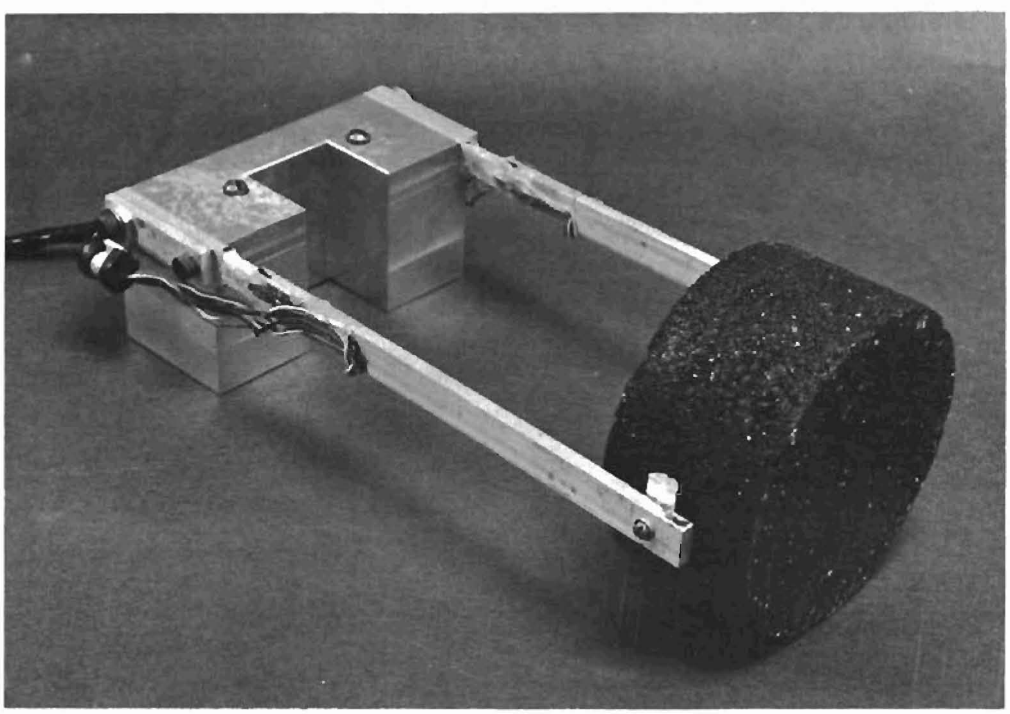


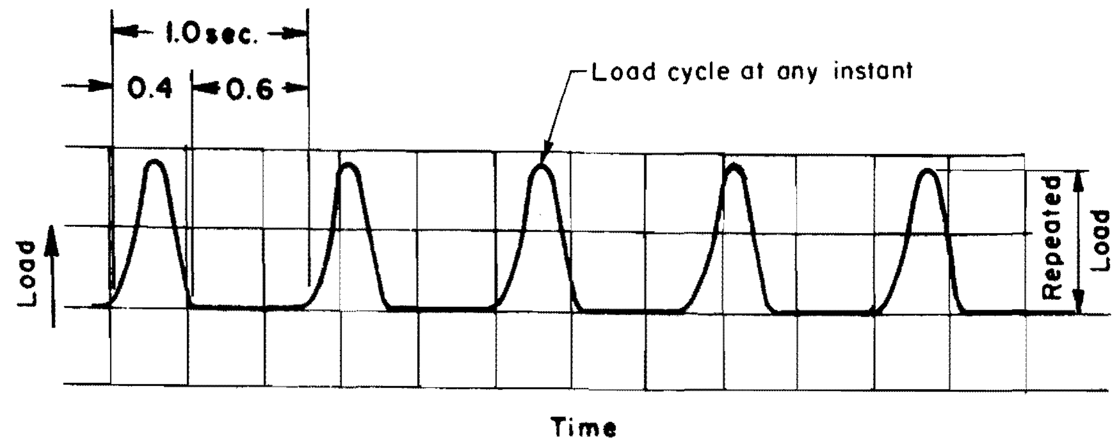
Fig 30. Lateral-strain measuring device.

In the case of repeated load tests, the horizontal and vertical deformations for selected cycles were recorded on a 2-channel strip chart, Hewlett-Packard Recorder Model 7402 A. A horizontal deformation transducer cosisting of two LVDT's (Trans Tek Series 350 with a working range of ± 0.050 inch and a mechanical travel of 0.14 inch) was used to monitor the horizontal deformation of the specimen which occurred during any given load cycle. A typical horizontal and vertical deformation versus time pattern, traced from an actual strip chart recording, is illustrated in Fig 31 along with the corresponding load-time pulse. The cumulative horizontal and vertical deformations were initially recorded on a pair of X-Y plotters used for the static test, but later a Data Logging System 12945 from Non-Linear Systems, Inc., was utilized. The deformations were indicated on a digital voltmeter and recorded on the system's paper tape printer. The instrument was calibrated to read vertical deformation in volts and horizontal deformation in millivolts. Readings were automatically taken and recorded every 10, 100, or 1000 cycles, depending on the expected fatigue life of the specimen. Each of the two recorders (a Hewlett-Packard Recorder and a Data Logging System) was powered by a Lambda Regulated Power Supply Model LL-903. A digital voltmeter was utilized to set the various voltage requirements. Typical plots of horizontal and vertical cumulative total deformations versus number of load repetitions are shown in Fig 32.

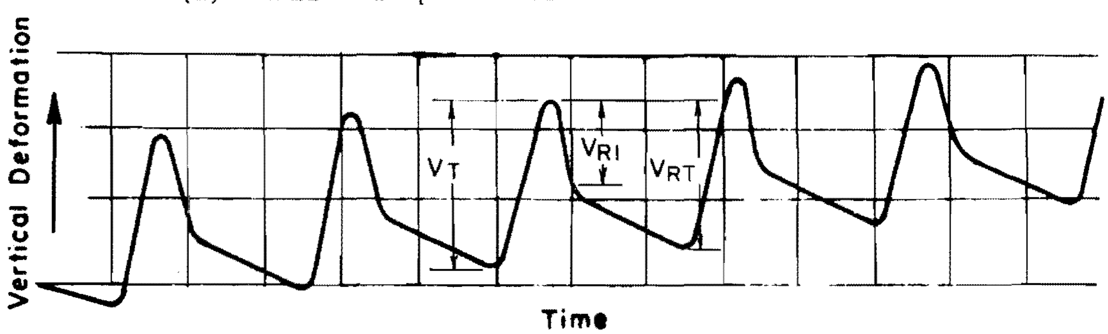
EXPERIMENTAL DESIGN

The basic experiment design was a full factorial which can be analyzed statistically (Refs 2, 19, and 25). Due to limitations of time and money, however, this design was modified during the testing phase of the study in order to minimize the number of tests. The factors and levels considered are briefly described in this section.

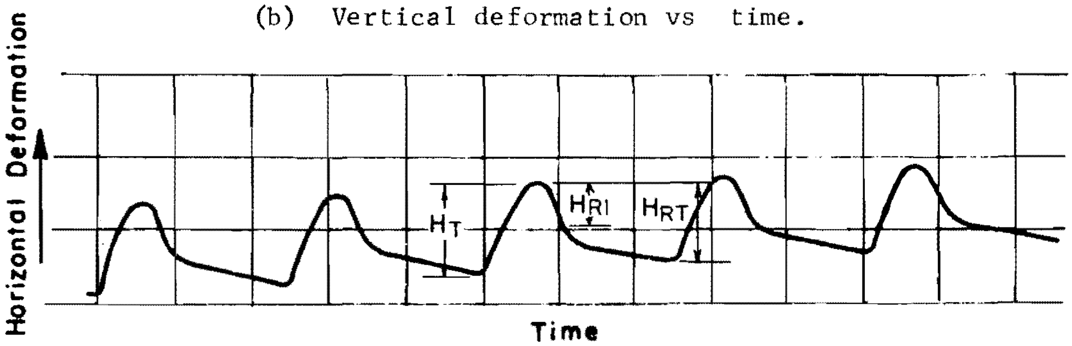
Previous studies at the CFHR and elsewhere had indicated that the fatigue life, modulus of elasticity, and Poisson's ratio of asphalt concrete are affected by such factors as stress, testing temperature, amount and type of asphalt, aggregate type and gradation, compaction effort, and mixing temperature. A completely randomized full-factorial design considering all



(a) Load-time pulse for indirect tensile test.



(b) Vertical deformation vs time.



(c) Horizontal deformation vs. time.

Fig 31. Typical load and deformation vs time relationship for repeated-load indirect tensile test.

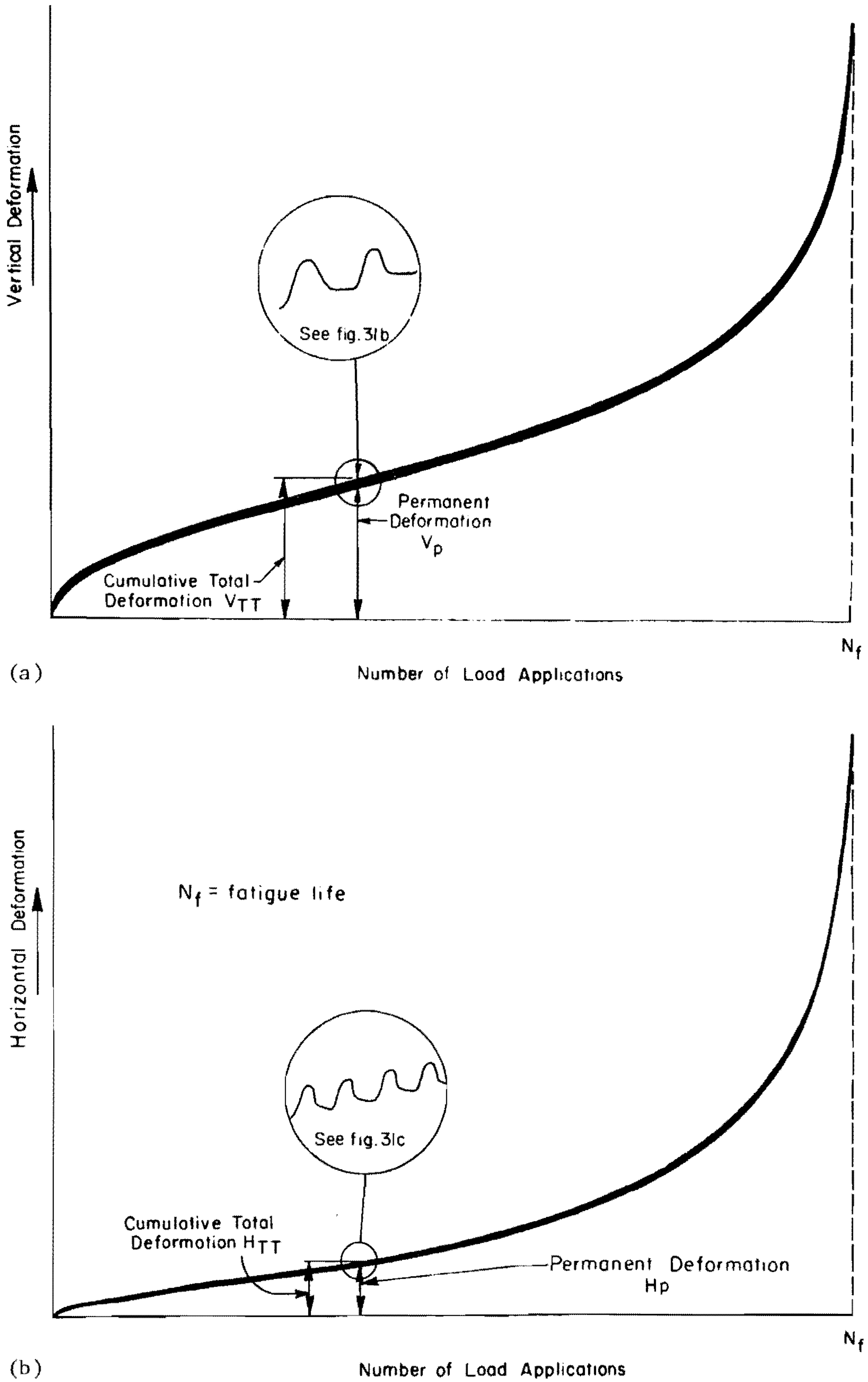


Fig 32. Relationships between number of load applications and vertical and horizontal deformation for the repeated-load indirect tensile test.

of these factors at three or more levels was not feasible due to the expense in money and time that the large number of specimens would require. Thus, careful consideration was given to the factors selected and the levels of such factors.

A full factorial design of the initial factors and levels considered would require 150 specimens if one specimen per cell were tested. However, due to the variations in fatigue testing and the probable variations associated with changes in elastic properties with repeated loading, it was decided to consider a full factorial design at four stress levels for tests conducted at room temperature, with between three and nine specimens per cell and also three specimens per cell at optimum asphalt content for tests at 50 and 100°F. It was hoped that the increased number of specimens per cell would give an estimate of the experimental error.

Because asphalt mixtures have high tensile strengths at low temperatures and low tensile strengths at high temperatures, it was not practical to maintain the same applied stress range for the three test temperatures. Figure 33 shows a graphical representation of the final design. The particular cells tested and the number of specimens tested in each cell are indicated. In the case of the static tests, three or more specimens were tested at each asphalt content (4 to 8 percent), testing temperature (50, 75, and 100°F), and aggregate type (limestone and gravel).

TESTING PROCEDURE

Prior to the test, all measuring devices were calibrated. To ensure noneccentric loading, an angle plate was utilized to align and center the specimen in the loading head. The upper platen of the die set was brought into light contact with the specimen and the load was monitored on the digital voltmeter. The horizontal device was placed on a rear platform with the arms in light contact with the specimen and then the arms were locked into position. Tests conducted at 50°F and 100°F were carried out in special temperature-controlled rooms.

Static Tests

In the case of the static tests, a preload of 20 pounds (less than 2 psi) was applied to the specimen to prevent impact loading and to minimize

Test Temp., °F	Stress Level, psi	Limestone					Gravel				
	Asphalt Content, %	4	5	6	7	8	4	5	6	7	8
50	72			3					3		
	96			3					3		
	120			3					3		
75	8				5*					5*	
	16	3	5	6	8	3	3	5	7	9	3
	24	5	5	7	5	5	5	5	7	9	5
	32	3	5	7	6	3	3	6	7	9	3
	40	4	5	7	8	3	3	5	7	8	3
100	8			3					3		
	16			3					3		
	24			3					3		

Number of specimens tested indicated in each cell.

*These specimens together with two specimens each at 48 psi and 56 psi stress levels for the limestone mixtures with 7% asphalt content tested at 75°F were obtained from a concurrent cumulative damage study (Ref 5).

Fig 33. Graphical representation of factorial experiment design for repeated-load tests.

the effect of seating the loading strip. The specimen was then loaded at a rate of two inches per minute. The load-vertical deformation and load-horizontal deformation plots were recorded on the pair of X-Y plotters.

Repeated-load Tests

In the repeated-load tests, also, a preload of 20 pounds was applied to the specimen. The additional amount of load required to produce the prescribed stress level 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. The above loading rate was selected after several trials using different loading rates, with primary consideration given to the capabilities of the recording instruments.

The vertical and horizontal deformation patterns per cycle were traced either continuously or at selected intervals, and the cumulative vertical and horizontal permanent deformation values were recorded at preset intervals. A limit switch value was set to automatically stop the test if the vertical deformation exceeded 0.25 inch in order to prevent damage to the recording devices and equipment. However, for nearly all specimens, a complete fracture occurred before reaching the limit switch value. The same test procedure was utilized for all tests at 50°F, 75°F, and 100°F. Typical deformation versus time relationships are presented in Figs 31 and 32.

DEFINITION OF TERMS

Both the tensile and elastic properties were estimated in various forms and the explanation of these terms is contained below. The calculations were based on the indirect tensile test theory as presented by Hondros (Ref 23) and contained in Appendix D. The working equations adapted from Hondros' equations, together with a list of computer programs utilized in the study, are also presented in Appendix D.

Figures 31 and 32, which illustrate typical traces of load, horizontal deformations, and vertical deformations of a specimen subjected to repeated application of load are essential for the following definitions.

Deformation

H_{RI}, V_{RI} = instantaneous resilient horizontal and vertical deformations, respectively.

H_{RT}, V_{RT} = total resilient horizontal and vertical deformations.

H_T, V_T = individual cycle total horizontal and vertical deformations.

H_P, V_P = cumulative permanent horizontal and vertical deformations.

H_{TT}, V_{TT} = cumulative total horizontal and vertical deformations.

Strain

ϵ_{RIH}, ϵ_{RIV} = instantaneous resilient horizontal and vertical strains, based on H_{RI} and V_{RI}, respectively.

ϵ_{RTH}, ϵ_{RTV} = total resilient horizontal and vertical strains, based on H_{RT} and V_{RT}, respectively.

ϵ_{TH}, ϵ_{TV} = individual total horizontal and vertical strains, based on H_T and V_T, respectively.

ϵ_{pH}, ϵ_{pV} = cumulative permanent horizontal and vertical strains, based on H_P and V_P, respectively.

ϵ_{TTH}, ϵ_{TTV} = cumulative total horizontal and vertical strains, based on H_{TT} and V_{TT}, respectively.

Poisson's Ratio

ν_{RI} = Poisson's ratio, based on instantaneous resilient strains ϵ_{RIH} and ϵ_{RIV}.

ν_{RT} = Poisson's ratio, based on total resilient strains ϵ_{RTH} and ϵ_{RTV}.

ν_T = Poisson's ratio based on individual total strains ϵ_{TH} and ϵ_{TV}.

ν_{TT} = Poisson's ratio based on cumulative total strains ϵ_{TTH} and ϵ_{TTV}.

Modulus

E_{RI} = instantaneous resilient modulus, based on ϵ_{RIH}, ϵ_{RIV}, and ν_{RI}.

E_{RT} = total resilient modulus, based on ϵ_{RTH}, ϵ_{RTV}, and ν_{RT}.

E_T = modulus of individual total deformation, based on ϵ_{TH}, ϵ_{TV}, and ν_T.

E_{TT} = modulus of cumulative total deformation, based on ϵ_{TTH}, ϵ_{TTV}, and ν_{TT}.

CHAPTER 4. ANALYSIS AND DISCUSSION OF STATIC TEST RESULTS

The primary purpose for conducting static tests was to obtain static tensile strengths, moduli of elasticity, and Poisson's ratios which could be used in the evaluation and analysis of some of the fatigue and dynamic test results. Table 5 summarizes the static test results, including the mean, standard deviation, and coefficient of variation for tensile strength, modulus of elasticity, and Poisson's ratio. The individual values of the above properties together with bulk density and air void content obtained for each specimen are contained in Appendix B.

TENSILE STRENGTH

Average strengths varied with asphalt content, testing temperature, and aggregate type and ranged between 7 psi and 584 psi for limestone mixtures and between 20 psi and 563 psi for gravel mixtures. The variation in strength was relatively small, with coefficients of variation generally less than about 10 percent (Table 5).

The strengths at 75°F were found to be generally lower than those previously reported for essentially the same mixtures (Ref 18). The difference in values, however, possibly can be explained by curing differences. Whereas specimens in this study were cured for 2 days at 75°F before testing, specimens in the previous study were cured for 14 days at 75°F. In addition, the location of the source of aggregates has changed slightly and this might affect the quality of the aggregates. However, the strengths were quite comparable to those obtained earlier for inservice blackbase and asphalt concrete specimens (Refs 39 and 55).

The relationships between indirect tensile strengths and asphalt contents for limestone and gravel mixtures tested at 50°, 75°, and 100°F are shown in Fig 34. While the maximum strengths for mixtures containing both aggregates were approximately the same, the optimum asphalt content for the gravel mixtures was slightly higher than that for the limestone mixtures, as shown by the line of optimums (Fig 34). The optimums at 50, 75, and 100°F

TABLE 5. SUMMARY OF STATIC TEST RESULTS

Temperature, °F	Aggregate	Asphalt Content, %	Number of Specimens	Tensile Strength			Modulus of Elasticity			Poisson's Ratio		
				Mean, psi	Std. Dev., psi	CV, %	Mean, psi	Std. Dev., psi	CV, %	Mean	Std. Dev.	CV, %
100	Limestone	4	3	27	3.7	13.6	26,200	4,320	16.5	0.63	0.09	14.4
		5	3	40	1.4	3.5	34,300	1,050	3.1	0.26	0.03	11.4
		6	3	48	0.5	1.0	33,600	2,260	6.7	0.72	0.15	20.8
		7	3	20	0.7	3.4	16,100	460	2.8	0.62	0.11	17.7
		8	3	7	2.2	32.4	5,000	1,270	25.4	0.56	0.09	16.0
	Gravel	4	3	22	0.1	0.3	29,100	2,100	7.2	0.46	0.01	2.2
		5	3	31	0.7	2.2	30,500	630	2.1	0.44	0.17	38.4
		6	3	39	3.4	8.8	48,900	1,580	3.2	0.44	0.07	16.0
		7	3	30	1.2	3.9	34,300	2,440	7.1	0.37	0.08	21.6
		8	3	20	0	0	18,000	5,090	28.2	0.57	0.17	29.6
75	Limestone	4	6	81	11.2	13.7	71,100	5,030	7.1	0.19	0.09	47.4
		5	6	125	6.5	5.2	99,200	12,190	12.3	0.08	0.06	72.0
		5½	3	145	8.1	5.6	115,500	12,640	11.0	0.08	0.02	26.1
		6	6	137	14.9	10.9	114,600	7,900	6.9	0.25	0.10	40.0
		7	13	109	6.5	5.9	82,900	13,020	15.7	0.36	0.10	27.4
		8	6	55	4.6	8.3	56,500	17,520	31.0	0.29	0.09	31.2
	Gravel	4	6	66	5.4	8.2	73,600	7,400	10.1	0.35	0.10	28.8
		5	6	104	7.0	6.7	104,800	12,830	12.2	0.21	0.05	24.4
		6	6	142	5.6	3.9	156,000	15,170	9.7	0.17	0.06	35.7
		6½	5	145	4.7	3.2	197,800	27,660	14.0	0.20	0.03	14.8
		7	13	134	9.2	6.9	160,800	18,270	11.4	0.26	0.06	23.2
		8	6	97	11.8	12.2	105,400	28,220	26.8	0.25	0.02	8.0
50	Limestone	4	3	203	25.3	12.5	243,200	16,990	7.0	-0.06	0.01	
		5	3	365	33.3	9.1	363,300	43,330	11.9	-0.07	0.04	
		6	3	584	57.0	9.8	595,200	8,900	0.7	-0.02	0.02	
		7	3	562	22.5	4.0	489,700	31,160	6.4	0.02	0.03	
		8	3	449	40.6	9.0	292,000	62,830	21.5	-0.03	0.03	
	Gravel	4	3	161	20.8	12.9	167,700	38,790	23.1	-0.11	0.03	
		5	3	312	17.6	5.6	314,900	25,720	8.2	-0.11	0.02	
		6	3	473	42.9	9.1	489,300	86,740	17.7	-0.09	0.02	
		7	3	563	13.9	2.5	652,100	92,280	14.2	-0.01	0.02	
		8	3	474	27.3	5.7	549,700	42,120	7.7	0.03	0.05	

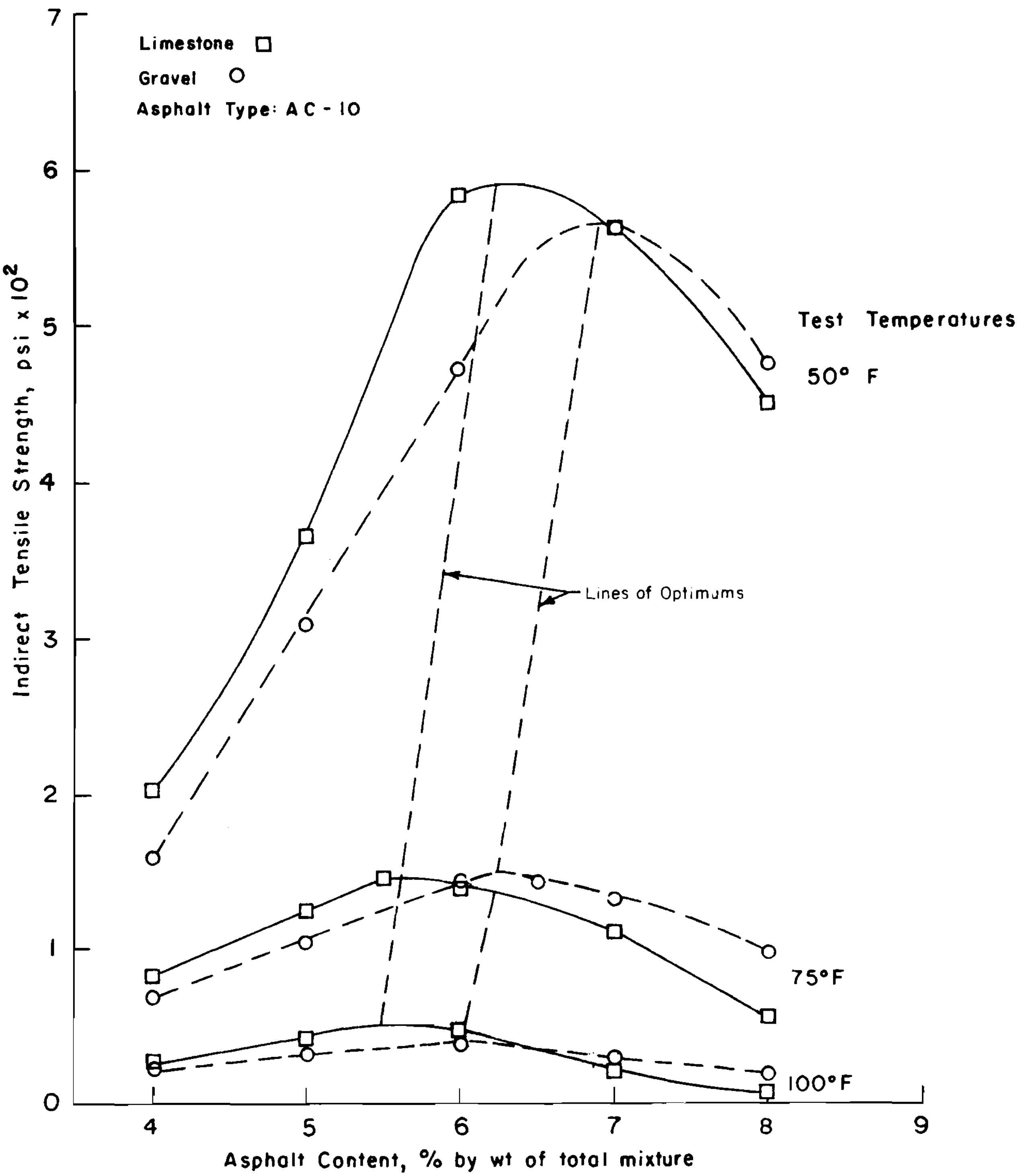


Fig 34. Relationships between average indirect tensile strengths and asphalt content for limestone and gravel mixtures.

were 6.9, 6.25, and 6.0 for the gravel mixtures and 6.25, 5.6, and 5.5 for the limestone mixtures. The same trend was noticed in an earlier study (Ref 18) for coarse graded mixtures. However, the same optimum asphalt content was obtained for both limestone and gravel mixtures in the case of medium gradation, and the optimum asphalt content for the limestone mixtures was slightly higher than for the gravel mixtures in the case of fine gradation. For both mixtures, the optimum asphalt content decreased with increasing temperature.

The effect of temperature on indirect tensile strength is illustrated in Fig 35, which shows that the higher the testing temperature, the lower the strength. This figure also shows that the change in strength between 50°F and 75°F is greater than the change between 75°F and 100°F. The effect of testing temperature noted above is similar to that reported by Hudson and Kennedy (Ref 26) even though the mixture and loading conditions were slightly different from those used in this study.

In addition, it may be noted from Figs 34 and 35 that the effect of asphalt content decreased with an increase in temperature. Thus, the effect of asphalt content was less important at the higher temperatures and, therefore, the selection of an optimum asphalt content would be less critical.

Since it was hoped that a relationship might exist between fatigue life and some function of strength, e.g., stress-strength ratio, an attempt was made to predict indirect tensile strength in terms of asphalt content, testing temperature, and aggregate type. Such predicted indirect tensile strengths could then be used to estimate strength values or stress-strength ratios in cases in which it would be difficult to obtain strength values by direct measurement. It was found that a logarithmic transformation substantially improved the homogeneity of variance (reduction of the chi-square value from 208 to 104 with 29 degrees of freedom), and that fact, along with theory, indicated that the analysis of variance and the regression analysis should be conducted using logarithmic transformations (Ref 2).

Table 6 shows the summary of the analysis of variance for the logarithm of tensile strength. As indicated in the table, the effects of all factors and all two-way interactions except the cubic effect of asphalt content and the quadratic effect of testing temperature were significant at a level of 5 percent or better. From this information multiple regression

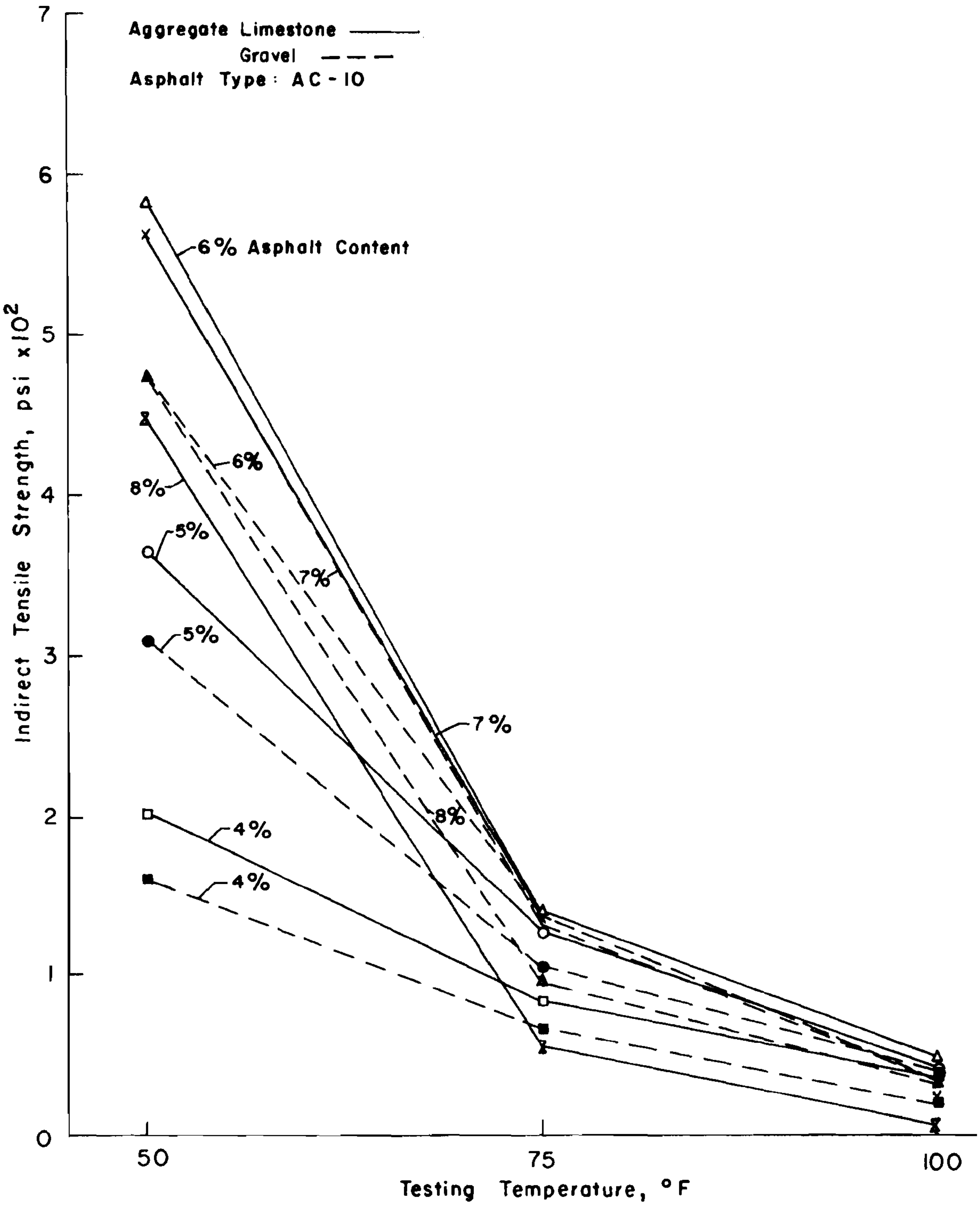


Fig 35. Effect of testing temperature on average indirect tensile strength.

TABLE 6. LEAST SQUARES ANALYSIS OF VARIANCE
FOR LOGARITHM OF TENSILE STRENGTH

Source of Variation	Degree of Freedom	Sum of Squares	Mean Squares	F Value	Significance Level α, %
Aggregate type	1	.012	.012	4.326	5.0
Asphalt content	4	1.753	.438	162.549	.05
Linear	1	.016	.016	6.064	2.5
Quad.	1	1.719	1.719	637.547	.05
Cubic	1	.002	.002	.799	NS*
Quard.	1	.016	.016	5.785	2.5
Temperature	2	20.879	10.440	3871.757	.05
Linear	1	20.877	20.877	7742.727	.05
Quad.	1	.002	.002	.787	NS*
Aggr. × asph.	4	.504	.126	46.687	.05
Aggr. × temp.	2	.058	.029	10.772	.05
Asph. × temp.	8	1.110	.139	51.450	.05
Remainder	112	.302	.003		
Total	133	24.833			

*NS - not significant at $\alpha \leq 10\%$

models were formulated and a regression analysis was conducted, yielding the following equation:

$$\begin{aligned} \text{Log } S_T = {} & 0.3202 + 0.9380\ A - 0.0116\ T - 0.0577\ A^2 \\ & - 0.0899\ GA + 0.0148\ GA^2 - 0.0003\ TA^2 \\ & (R^2 = 0.98,\ S_e = 0.06) \end{aligned} \tag{4.1}$$

where

S_T = predicted indirect tensile strength, psi,
A = asphalt content, percent by weight of total mixture,
T = testing temperature, °F, and
G = coded value for aggregate (limestone = 0, gravel = 1).

The above expression, with a coefficient of multiple determination R^2 of 0.98 and a standard error of estimate S_e of 0.06, accounted for all but 2 percent of the total variation in the logarithm of tensile strength. It must be remembered, however, that Eq 4.1 was obtained for aggregates consisting of limestone and gravel, asphalt contents ranging between 4 and 8 percent, and testing temperatures of 50, 75, and 100°F. Therefore, any use of the equation for other conditions should be made with caution.

In an earlier study (Ref 18), a similar attempt was made to predict tensile strength based on findings on laboratory specimens and the following equation was obtained:

$$\begin{aligned} S_T = {} & 150.8 - 5.027(B - 4.0) + 26.037(C - 910) - 12.691(D - 7.0) \\ & + 0.574(G - 250.0) - 10.929(B - 4.0)(D - 7.0) + 3.572(A - 1.0) \\ & (B - 4.0)(D - 7.0) - 0.0688(B - 4.0)(G - 250.0) - 0.0750 \\ & (B - 4.0)(D - 7.0)(G - 250.0) - 3.2775(B - 4.0)^2 - 11.545 \\ & (D - 7.0)^2 \end{aligned} \tag{4.2}$$

with an R^2 of 0.78 and an S_e value equal to 28. The meanings of the symbols together with the levels of factors considered in arriving at the above regression equation are shown in Table 7.

Since Eq 4.1 from this study and Eq 4.2 from a previous study were obtained for essentially the same mixtures utilizing the same test method, it was felt that they should predict comparable strength values, even though

TABLE 7. LEVELS OF FACTORS USED IN REGRESSION EQUATIONS (REF 18).

Factor	Description	Level
A - Aggregate type	Limestone	A(-1) = 0
	Seguin gravel	A(+1) = 2
B - Aggregate gradation	Fine	B(-1) = 2
	Medium	B(0) = 4
	Coarse	B(+1) = 6
C - Asphalt viscosity	AC-5	C(-1) = 8.5
	AC-10	C(0) = 9.0
	AC-20	C(+1) = 9.7
D - Asphalt content	Low-low	D(-2) = 4.0
	Low	D(-1) = 5.5
	Medium	D(0) = 7.0
	High	D(+1) = 8.5
	High-high	D(+2) = 10.0
F - Mixing temperature	Low	F(-1) = 250
	Medium	F(0) = 300
	High	F(+1) = 350
G - Compaction temperature	Low	G(-1) = 200
	Medium	G(0) = 250
	High	G(+1) = 300
H - Curing temperature	Low	H(-1) = 40
	Medium	H(0) = 75
	High	H(+1) = 110

different factors or variables were used in the equations. A comparison of predicted indirect tensile strengths from the two equations is shown in Fig 36, from which it can be seen that the equations produce nearly identical results. Thus, it seems either equation can be used to predict tensile strength, depending on available information about the variables. However, since testing temperature, which was considered in this study, is a measure of a major environmental factor affecting tensile strength and because a higher R^2 was obtained in the resulting regression equation, the use of Eq 4.1 might be more practical.

MODULUS OF ELASTICITY

Average values of static modulus of elasticity varied with asphalt content, testing temperature, and aggregate type and ranged between 5,000 psi and 595,000 psi for limestone mixtures and between 18,000 psi and 652,000 psi for gravel mixtures. The computed coefficients of variation ranged between 1 percent and 31 percent for both limestone and gravel mixtures at the various asphalt contents.

The effect of asphalt content on the average modulus is illustrated in Fig 37 in which optimum asphalt contents for maximum modulus is evident. The maximum modulus at the optimum asphalt content was larger for the gravel mixtures. However, for asphalt contents less than about 6 percent, the moduli for the gravel mixtures were approximately equal to or, in the case of 50°F, slightly less than the moduli for limestone mixtures whereas for asphalt contents greater than about 6 percent, the moduli for gravel mixtures were much larger than those for limestone mixtures. As with tensile strength, the optimum asphalt content for the maximum modulus of elasticity was slightly larger for the gravel mixtures than for the limestone mixtures (Fig 37). The optimums at 50, 75, and 100°F were 7.0, 6.5, and 6.3 for the gravel mixtures and 6.2, 5.8, and 5.75 for the limestone mixtures, as shown by the line of optimums.

The effect of temperature on modulus values is shown in Fig 38, in which moduli increased as temperature decreased. The increase in modulus between 75°F and 50°F was much greater than the corresponding increase between 100°F and 75°F. The effects of temperature on modulus values obtained in this study are similar to those obtained for tangent modulus of vertical

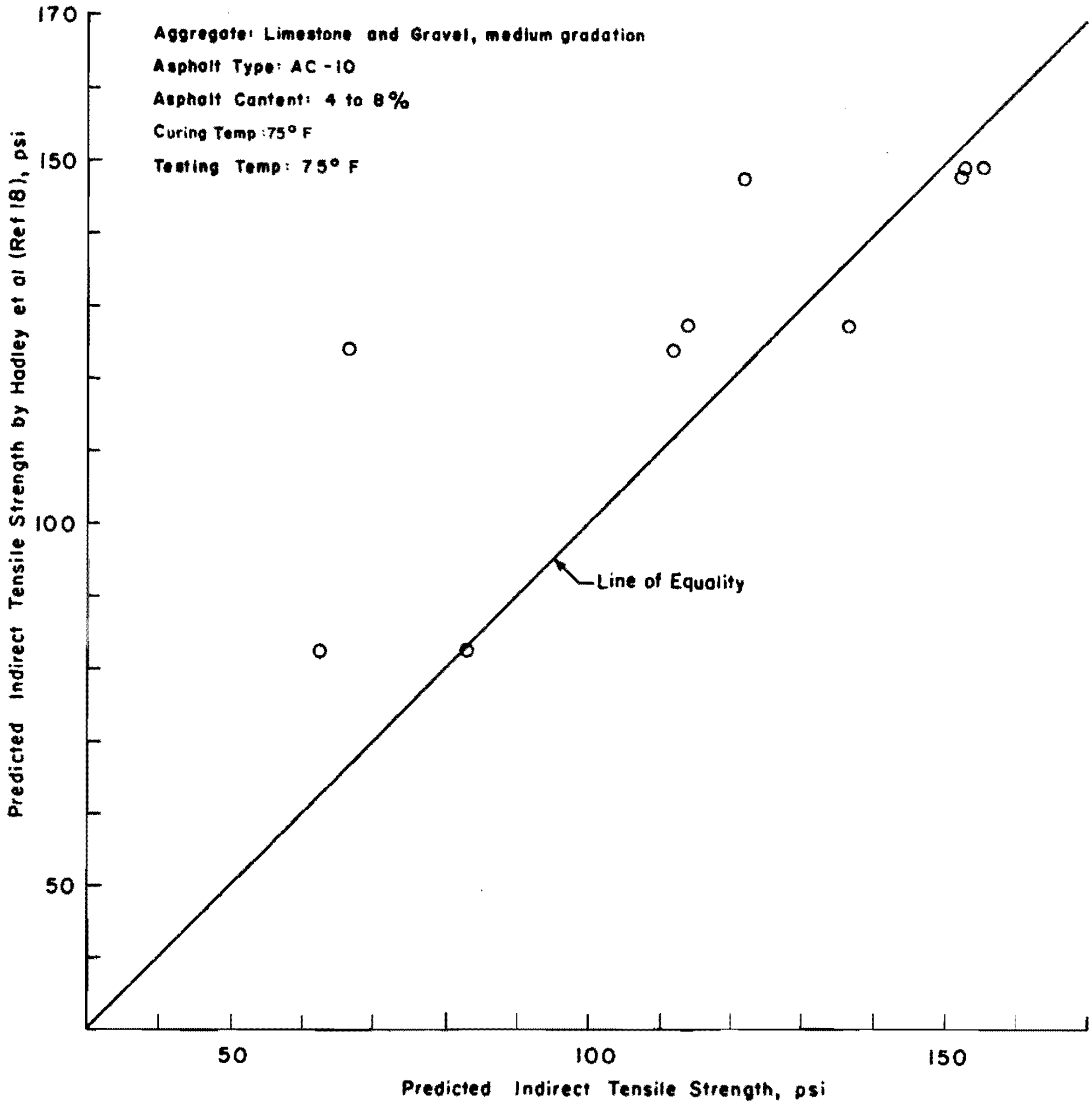


Fig 36. Comparison of predicted indirect tensile strengths from Eq 4.1 (this study) and Eq 4.2 (Hadley et al, Ref 18).

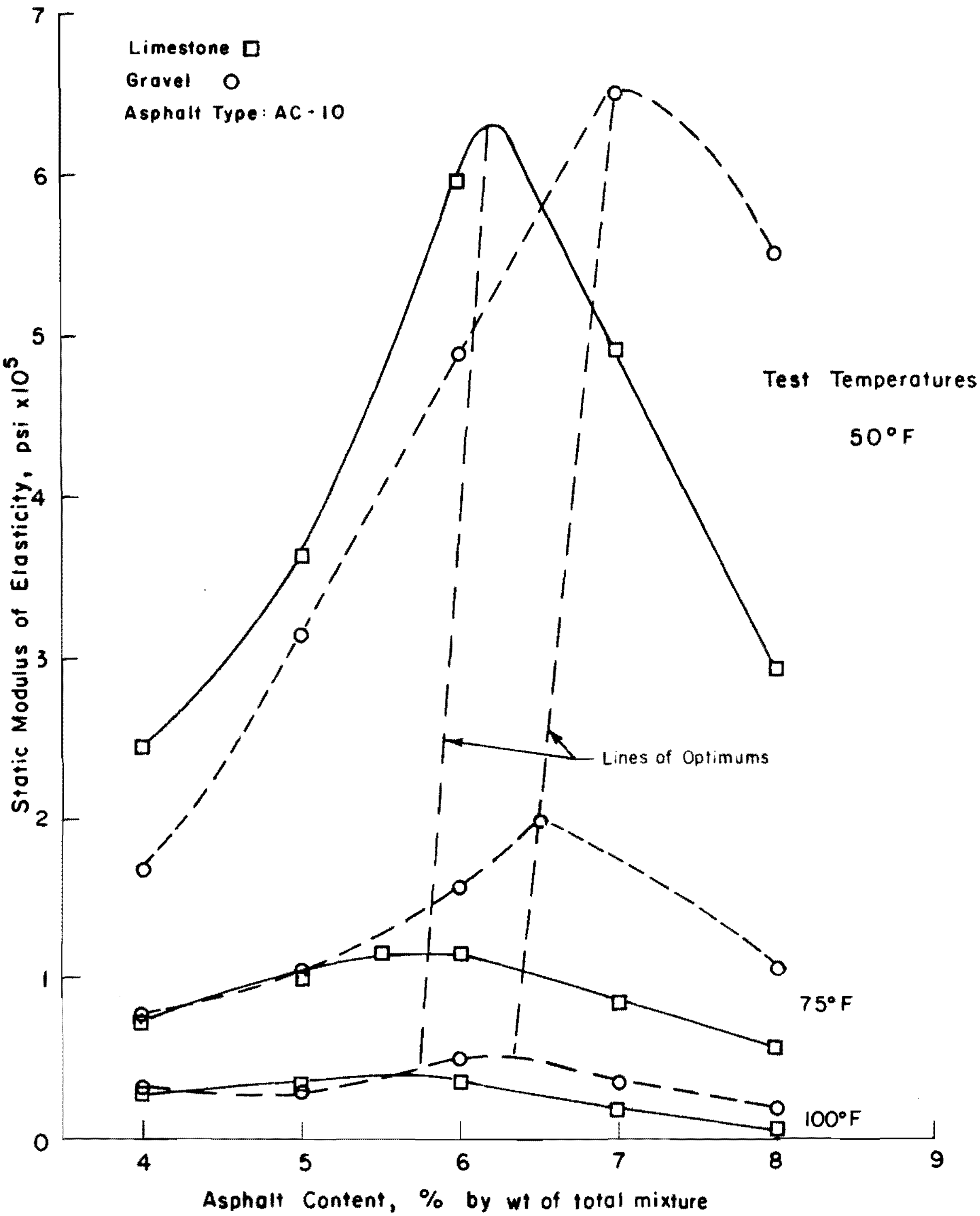


Fig 37. Relationships between average static modulus of elasticity and asphalt content for limestone and gravel mixtures.

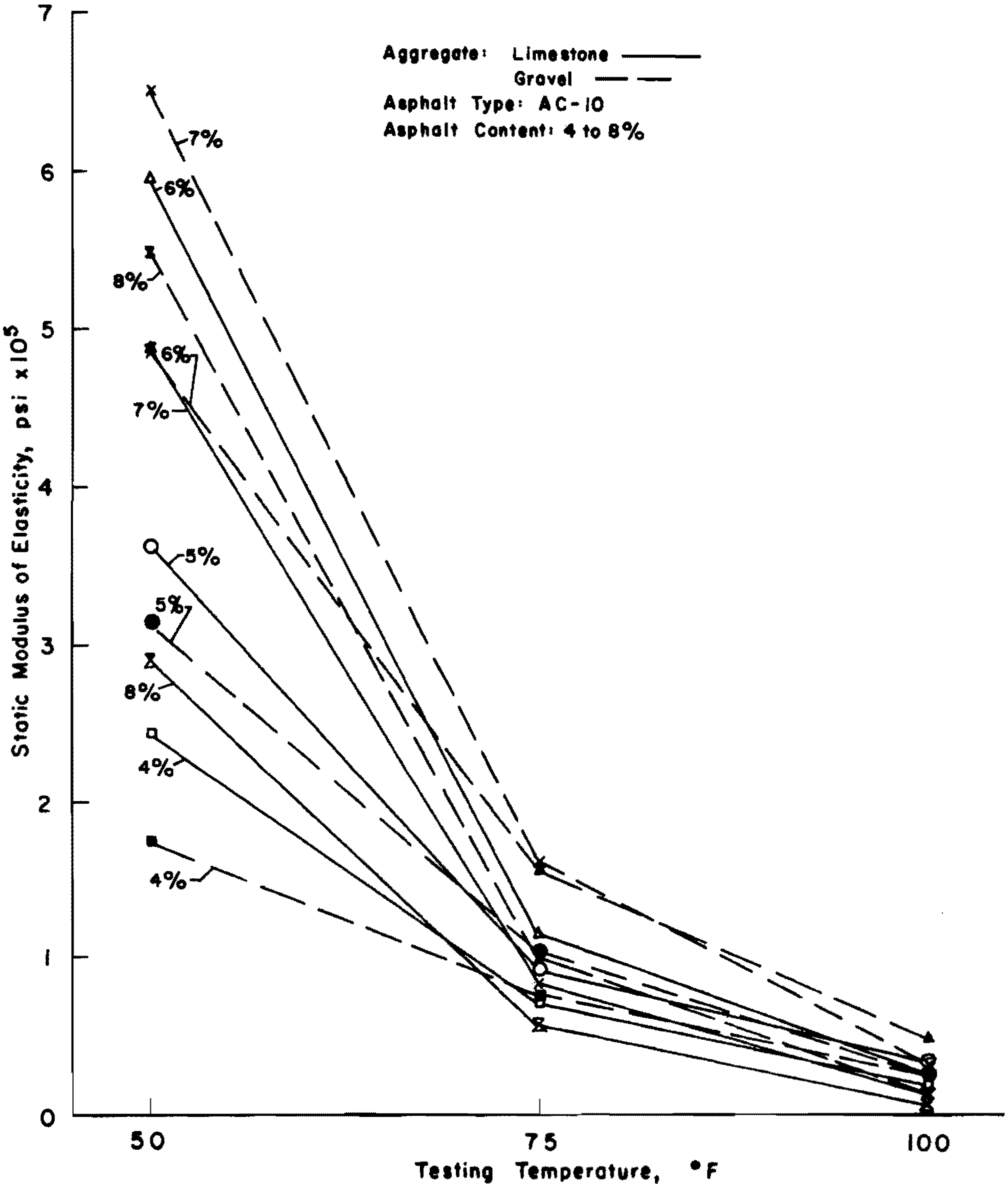


Fig 38. Effect of testing temperature on average static modulus of elasticity.

deformation in an earlier study (Ref 26) and for strength in this study. As in the case of tensile strength, the effect of asphalt content decreased with an increase in temperature and, thus, the selection of an optimum asphalt content would be less critical at high temperatures.

Comparison of moduli of elasticity at 75°F indicates that values obtained from this study are much lower than those obtained for the same mixtures in a previous study (Ref 18). As was the case for tensile strength, this difference may be due to the difference in the curing times. The values, however, seem to fall within the range previously obtained for inservice blackbase and asphalt concrete specimens (Refs 39 and 55).

POISSON'S RATIO

Results from this study indicate that for the range of asphalt content considered there was no consistent relationship between asphalt content and Poisson's ratio for mixtures containing both limestone and gravel aggregates (Fig 39). However, results presented in an earlier study on the same mixtures (Ref 18) indicated that for limestone mixtures Poisson's ratio decreased very slightly with increased asphalt content while, for gravel mixtures, the effect of Poisson's ratio was more significant, with an increase in asphalt content producing an increase in Poisson's ratio. Testing temperature was also found to affect Poisson's ratio, with higher temperatures producing higher Poisson's ratios.

Poisson's ratio varied very widely, ranging between negative values, for tests conducted at 50°F, and values near or greater than 0.5, for tests conducted at 100°F. Comparison with values presented by Hadley et al (Ref 18) indicated that, for the gravel mixtures, mean Poisson's ratios obtained from this study were generally higher, whereas, for limestone mixtures, values were higher only at asphalt contents greater than about 6 percent. For lower asphalt contents, Poisson's ratios in this study were lower than the corresponding values presented in Ref 18. For tests conducted at 75°F, the range of mean Poisson's ratios compared well with those previously obtained for the same mixtures (Ref 18) and also those obtained for inservice blackbase and asphalt concrete mixtures (Refs 39 and 55).

At the approximately optimum asphalt content for strength and modulus, and at a testing temperature of 75°F, Poisson's ratios were 0.18 for gravel and 0.14 for limestone.

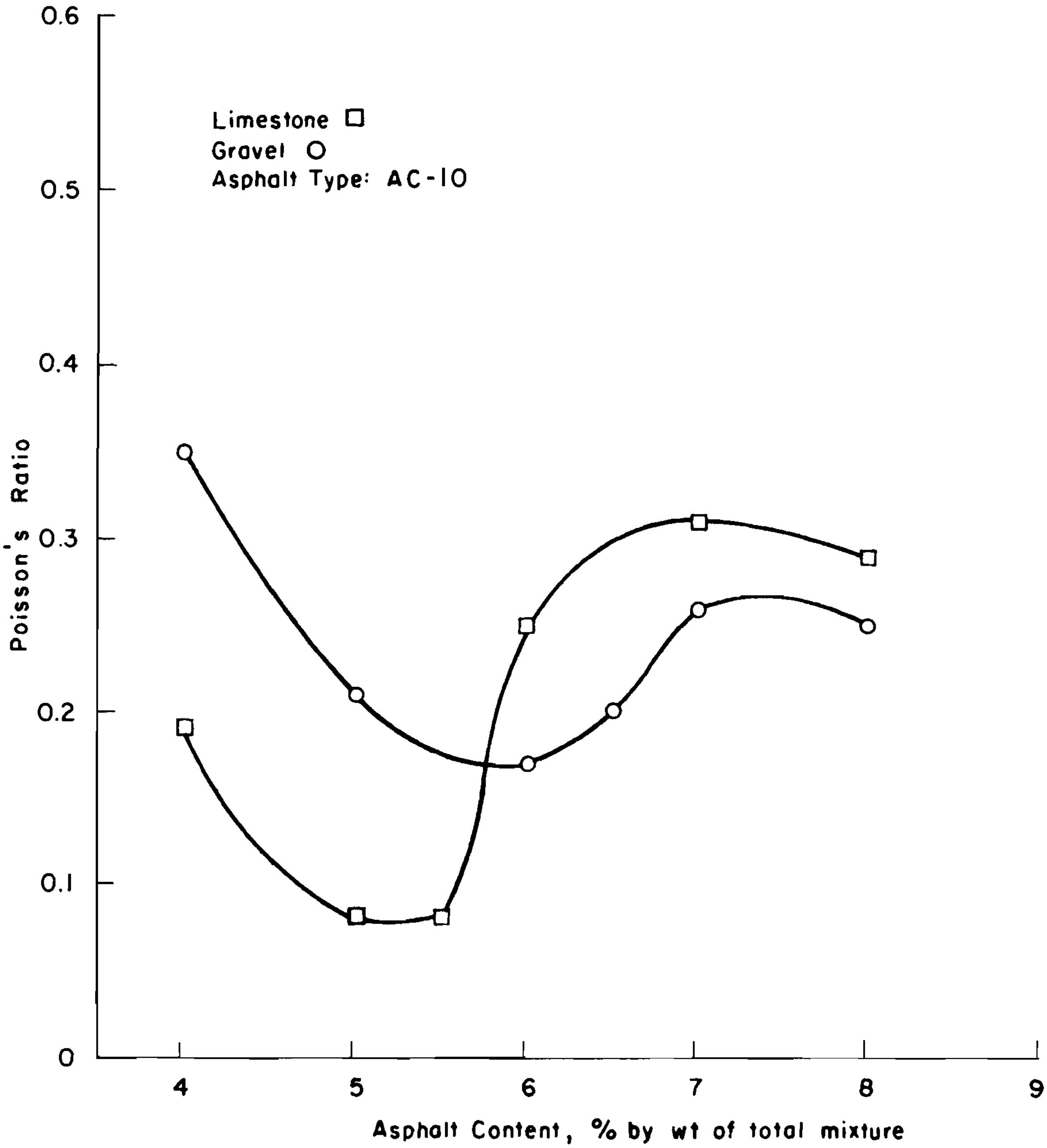


Fig 39. Relationships between average Poisson's ratio and asphalt content for limestone and gravel mixtures tested at 75°F.

DISCUSSION OF OPTIMUM ASPHALT CONTENTS

From the results of static tests, various optimum asphalt contents for different properties were obtained. As illustrated in Fig 40, the optimum asphalt contents for indirect tensile strength and static modulus of elasticity were affected by both aggregate type and testing temperature. The optimum asphalt content decreased with increasing testing temperature and was lower for the limestone mixtures than for the gravel mixtures. As indicated in Fig 40, the optimums for density, which are often used for design purposes, were generally higher than the optimums for indirect tensile strength and static modulus of elasticity. The only exception was for the gravel mixtures tested at 50°F.

Although the above observations might be peculiar to the particular mixtures and conditions of test, it should be noted that optimum asphalt content for density is not necessarily the same as the optimums for strength and modulus.

RELATIONSHIPS BETWEEN DEPENDENT STATIC PROPERTIES

Results from this study indicated a possible correlation between indirect tensile strength and static modulus of elasticity.

Such a relationship would be useful for those agencies which are not equipped to conduct the indirect tensile test with deformation measurements, such as the district laboratories of the Texas Highway Department. A regression analysis for modulus of elasticity and indirect tensile strength with modulus of elasticity as the dependent variable showed that an approximately linear relationship exists between the logarithm of modulus of elasticity and the logarithm of indirect tensile strength for the mixtures and the range of temperatures considered (50°, 75°, and 100°F), as follows:*

$$\log E_S = 2.94 + 1.03 \log S_T \qquad (4.3)$$
$$(R = 0.97,\ S_e = 0.11)$$

*Regression analysis for E_S and S_T without logarithmic transformation produced a correlation coefficient of 0.91.

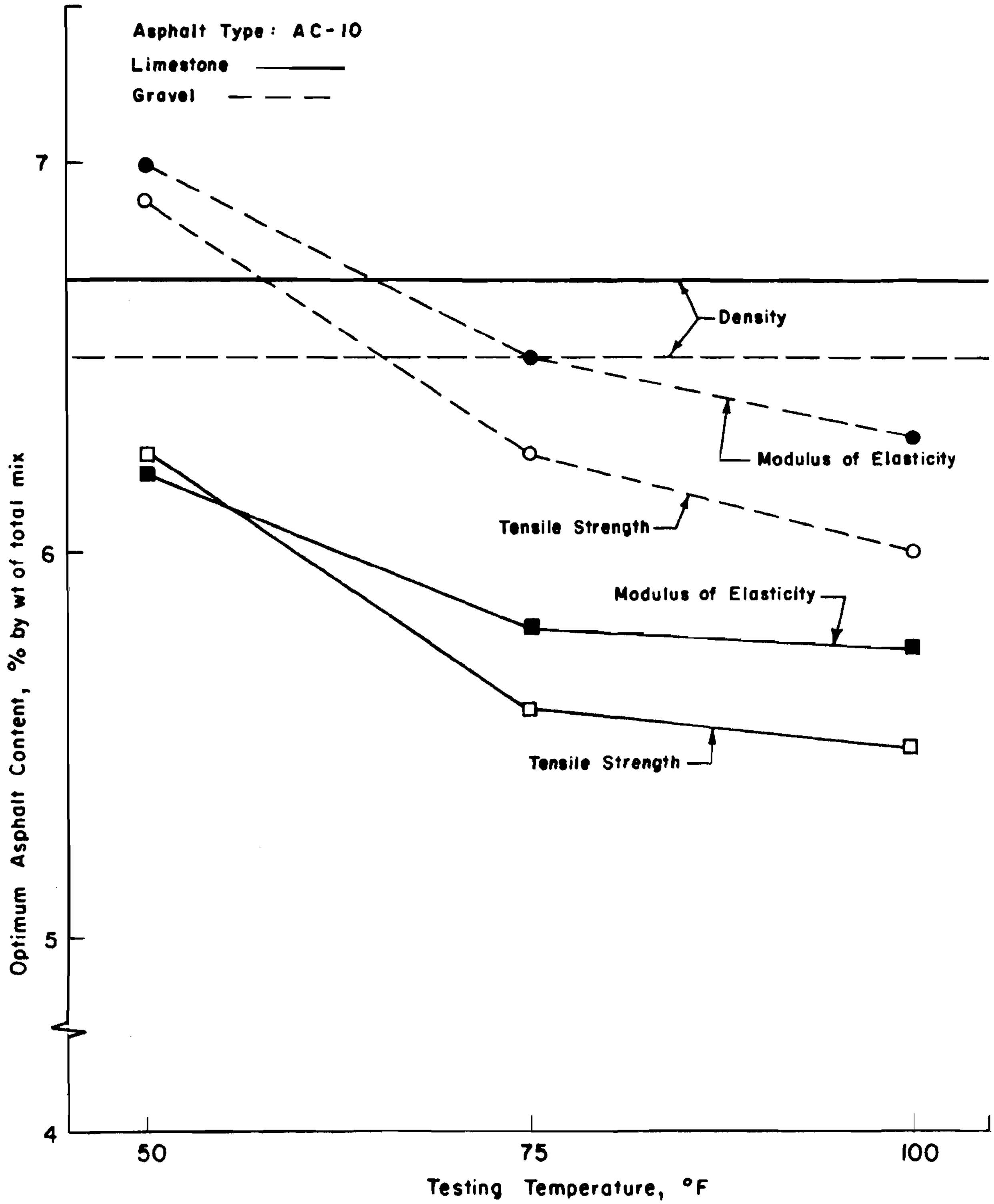


Fig 40. Comparison of optimum asphalt contents for density and static properties.

where

E_S = static modulus of elasticity, psi, and
S_T = indirect tensile strength, psi.

The above relationship was obtained with a correlation coefficient R of 0.97. The 95 percent confidence interval for R was between 0.96 and 0.98. Equation 4.3 was obtained for values of E_S ranging between about 4,000 psi and 760,000 psi and for S_T ranging between about 6 psi and 600 psi. In the regression analysis, it was assumed that the combination (E_S, S_T) is a random sample of a population of combinations from the ranges given above. Figure 41 shows a plot of the logarithm of E_S vs the logarithm of S_T with Eq 4.3 superimposed.

Since the correlation between E_S and S_T was strong, it was felt that Eq 4.3 possibly could be used to estimate E_S in terms of S_T . To test this, the measured S_T values for inservice blackbase and asphalt concrete mixtures (Ref 55) were used with Eq 4.3 to estimate E_S values. The predicted E_S values were then compared with the measured E_S values, as shown in Fig 42. Since the predicted values compared fairly well with the measured values, it is felt that Eq 4.3 can be used to estimate modulus of elasticity in terms of measured indirect tensile strength. However, the equation should be used only when it is not possible or practical to obtain measured values and should not be used as justification for not making the necessary deformation measurement. In addition, it is recommended that additional values for a variety of conditions be obtained in order to improve the relationship and extend its inference.

Additional analyses indicated that no strong correlations existed between the following dependent properties for tests conducted at 75°F:

(1) modulus of elasticity and bulk density,
(2) modulus of elasticity and air void content,
(3) indirect tensile strength and bulk density, and
(4) indirect tensile strength and air void content.

Hadley et al (Ref 18) had also concluded that there was no correlation between modulus of elasticity and density nor between tensile strength and density for the same mixtures tested at 75°F.

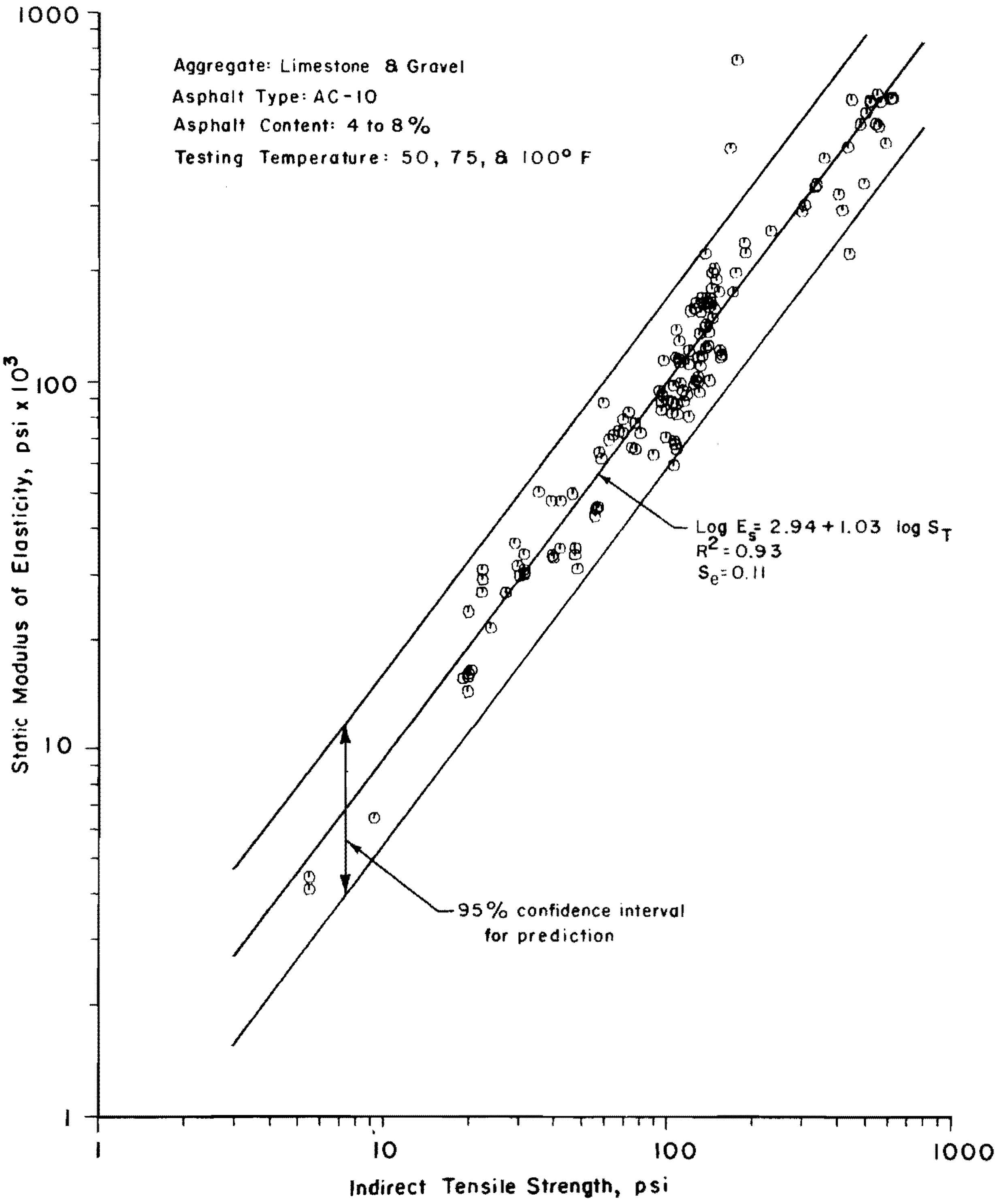


Fig 41. Logarithmic relationship between static modulus of elasticity and indirect tensile strength.

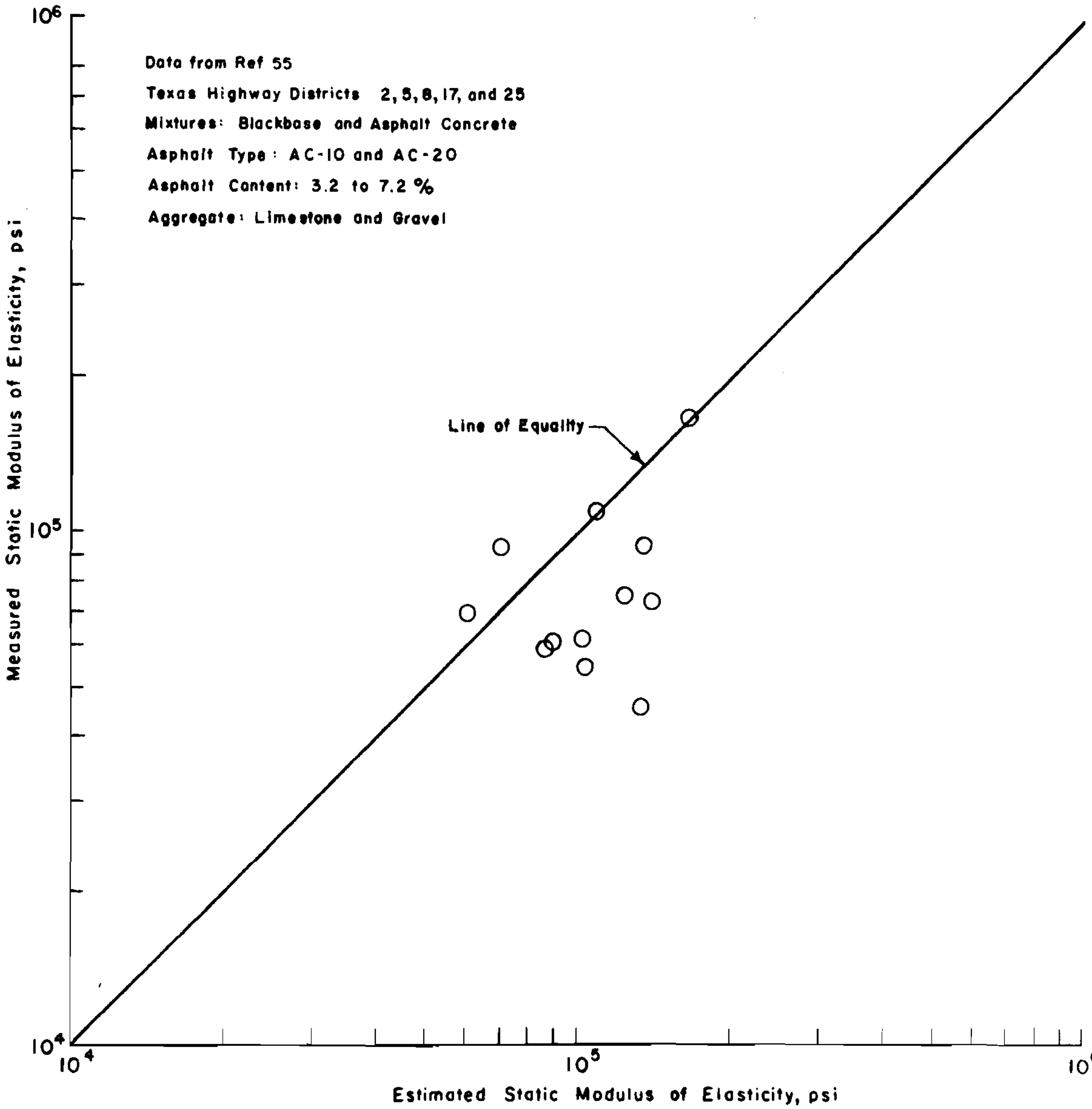


Fig 42. Relationship between measured and predicted modulus of elasticity.

CHAPTER 5. FATIGUE TEST RESULTS: PRESENTATION, ANALYSIS, AND DISCUSSION

In this chapter the results of fatigue tests together with the analysis and the results from such analysis are presented. Since the controlled-stress mode of loading was utilized in this study, fatigue lives were estimated on the basis of fracture lives, that is, the number of load applications required to completely fracture a specimen.

FATIGUE LIFE

The fatigue test results for all the mixtures and test temperatures are summarized in Table 8, which contains the arithmetic mean, standard deviation, coefficient of variation, and geometric mean, or log mean, of the fatigue lives for each asphalt content and stress level. The fatigue lives, bulk density, and air void content for individual specimens are presented in Appendix C.

Variations in Fatigue Life

Table 8 indicates that the coefficients of variation ranged from 4 to 70 percent with most values less than about 30 percent. These values are generally lower than those previously reported for beam and other fatigue tests (Refs 11, 48, and 75). The generally low values may be due to several reasons, such as uniformity in specimen manufacture and the fact that the indirect tensile test generally produces less variations than other test methods (Refs 26 and 34).

Relationships with Applied Stress

As in previous studies, a linear relationship was found to exist between the logarithm of applied stress and the logarithm of fatigue life.

The individual relationships between the logarithm of applied stress and the logarithm of fatigue life for the various mixtures containing limestone and gravel tested at 50, 75, and 100°F are shown in Figs 43 and 44. The relationships between fatigue life and applied stress are expressed

TABLE 8. SUMMARY OF FATIGUE RESULTS

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	No. of Specimens	Fatigue Life-N_f Mean, cycles	Standard Deviation	CV, %	'Log' Mean
50	Limestone	6	72	3	5,405	889	16.4	5,352
			96	3	1,667	193	11.6	1,660
			120	3	800	58	7.3	799
	Gravel	6	72	3	4,331	769	17.6	4,288
			96	3	1,035	135	13.0	1,029
			120	3	302	24	7.0	301
75	Limestone	4	16	3	625	113	18.1	618
			24	5	167	31	18.6	165
			32	3	109	25	22.9	107
			40	4	53	31	58.5	47
		5	16	5	3,334	626	18.8	3,280
			24	5	895	231	25.8	870
			32	5	456	67	14.7	452
			40	5	231	38	16.5	228
		6	16	6	8,029	1,342	16.7	7,938
			24	7	2,364	764	32.3	2,264
			32	7	883	133	15.1	875
			40	7	434	65	15.0	429
		7	8	5	219,720	58,296	26.5	214,536
			16	8	8,344	2,148	25.7	8,117
			24	5	1,643	239	14.5	1,629
			32	6	621	135	21.7	610
			40	8	360	81	22.5	353
			48	2	188	25	13.3	187
			56	2	129	13	10.1	129
		8	16	3	3,957	612	15.5	3,926
			24	5	1,138	181	15.9	1,125
			32	3	388	82	21.1	382
			40	3	194	16	8.2	194
	Gravel	4	16	3	333	13	3.9	333
			24	5	101	31	30.7	97
			32	3	40	14	35.0	39
			40	3	21	5	23.8	21
		5	16	5	1,509	139	9.2	1,503
			24	5	443	114	25.7	430
			32	6	188	65	34.6	179
			40	5	103	16	15.5	102
		6	16	7	8,063	2,503	31.0	7,701
			24	7	1,605	389	24.2	1,565
			32	7	809	220	27.2	780
			40	7	298	62	20.8	293
		7	8	5	350,232	141,580	40.4	329,035
			16	9	6,309	2,247	35.6	5,956
			24	9	1,532	779	50.8	1,375
			32	9	563	280	49.7	503
			40	8	425	193	45.4	383
		8	16	3	4,413	3,079	69.8	3,619
			24	5	875	454	51.9	800
			32	3	378	175	46.3	344
			40	3	209	20	9.6	208
100	Limestone	6	8	3	2,151	405	18.8	2,123
			16	3	328	35	10.7	326
			24	3	116	14	12.1	115
	Gravel	6	8	3	1,194	380	31.8	1,148
			16	3	177	32	18.1	176
			24	3	58	7	12.1	58

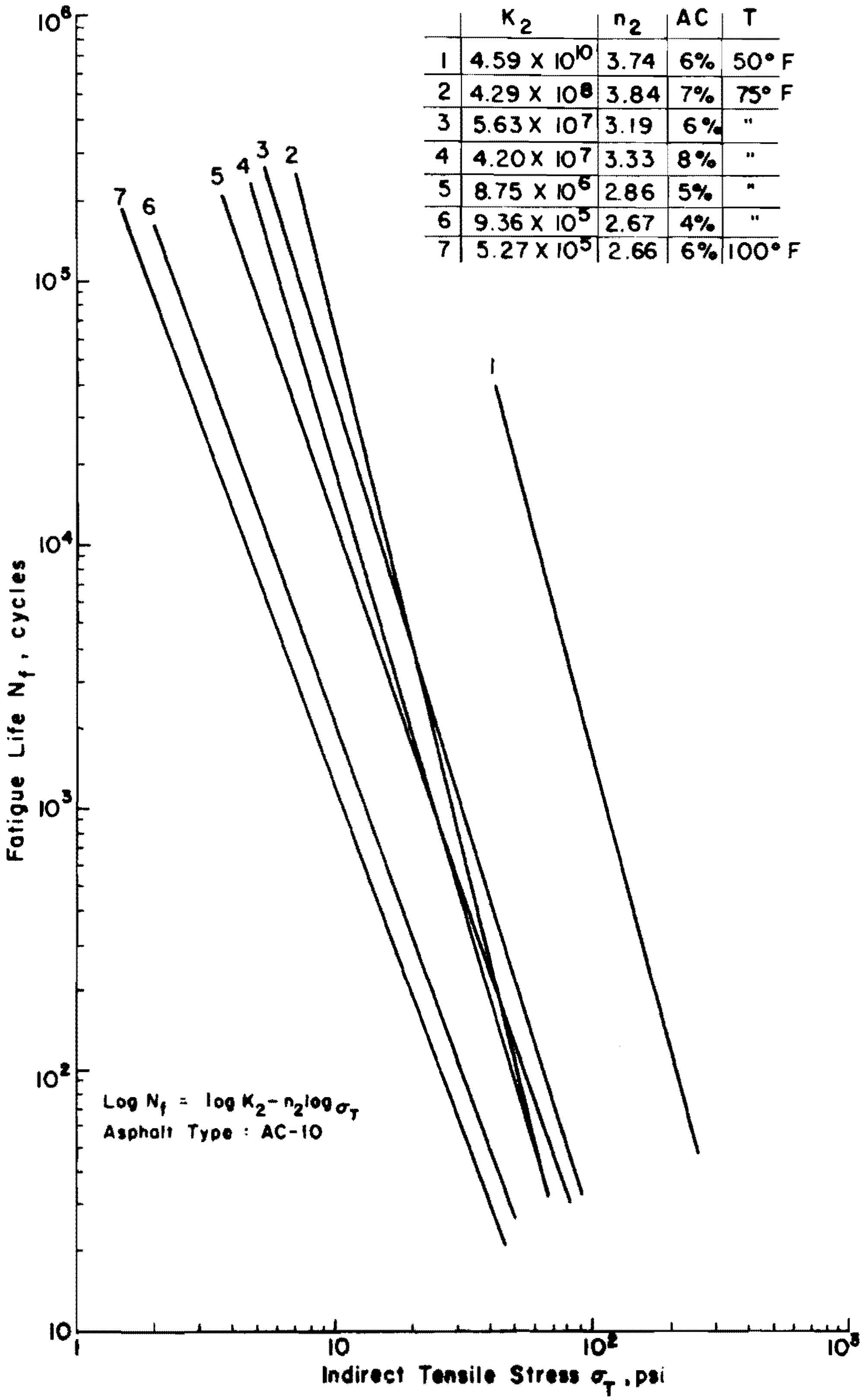
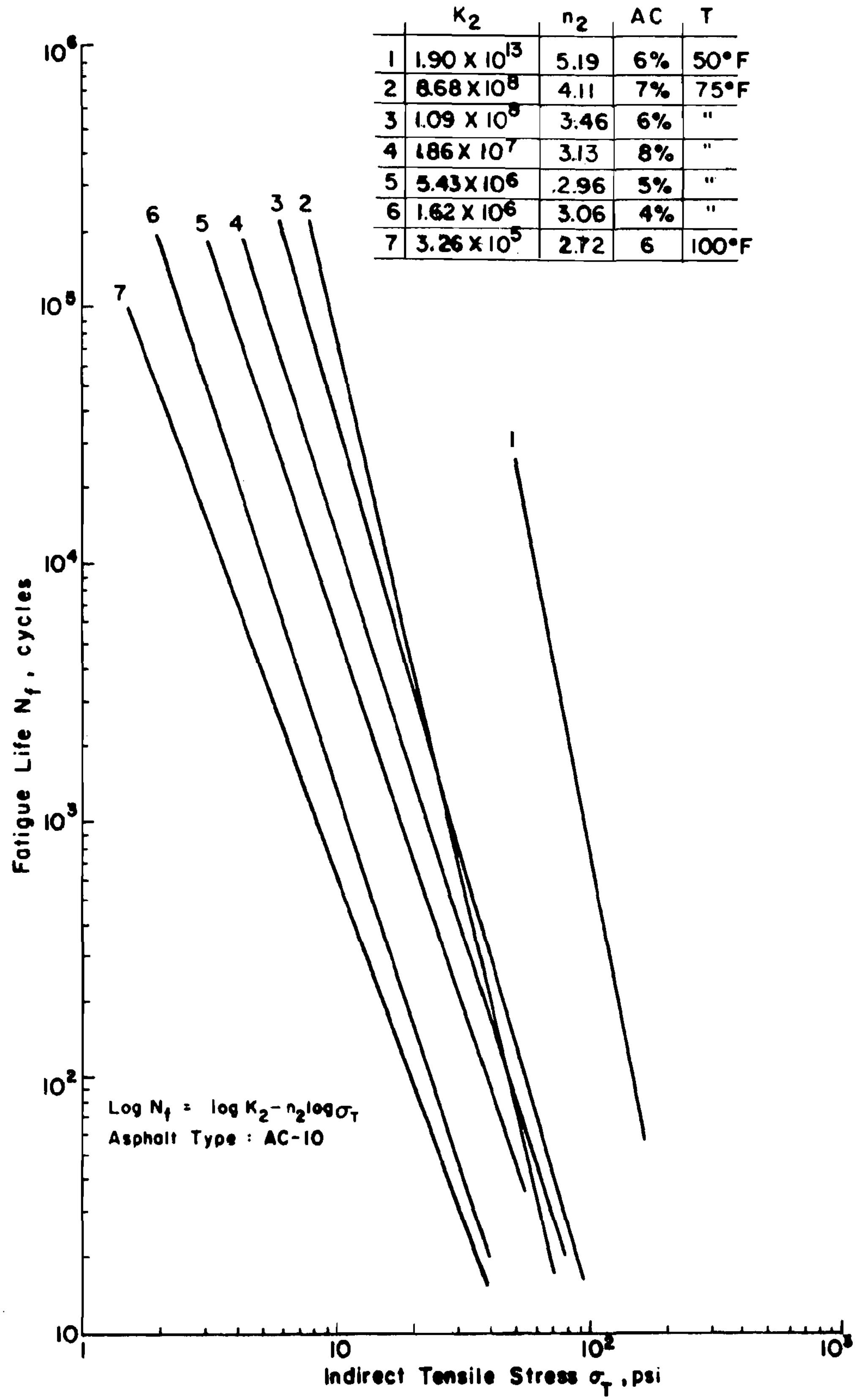


Fig 43. Relationships between fatigue life and applied tensile stress for limestone mixtures.



	K_2	n_2	AC	T
1	1.90×10^{13}	5.19	6%	50°F
2	8.68×10^{8}	4.11	7%	75°F
3	1.09×10^{8}	3.46	6%	"
4	1.86×10^{7}	3.13	8%	"
5	5.43×10^{6}	2.96	5%	"
6	1.62×10^{6}	3.06	4%	"
7	3.26×10^{5}	2.72	6	100°F

Fig 44. Relationships between fatigue life and applied tensile stress for gravel mixtures.

in the form of Eq 2.5:

$$N_f = K_2 \left(\frac{1}{\sigma}\right)^{n_2}$$

where

N_f = fatigue life,

σ = applied stress, and

K_2, n_2 = constants, depending on mixture properties and testing temperature.

By performing regression analysis, the values of the constants K_2 and n_2 together with correlation coefficients and standard errors of estimate were obtained and are summarized in Table 9. The values of K_2 and n_2 are also indicated for each of the relationships in Figs 43 and 44. For the individual limestone and gravel mixtures, the values of K_2 varied between 3.26×10^5 and 1.90×10^{13} while n_2 varied between 2.66 and 5.19, depending on mixture and temperature.

The values of K_2 and n_2 obtained from this study were compared to values obtained from controlled-stress flexural and axial load tests on other mixtures (summarized in Appendix A). The summary of values for K_2 and n_2 indicates that K_2 generally has ranged from 8.0×10^7 to 8.2×10^{21} while n_2 ranged from 1.85 to 7.1. It is apparent that while the values of n_2 compare favorably with those obtained for most mixtures, irrespective of test method, the values of K_2 are significantly smaller.

Previous findings have shown that because of the biaxial state of stress developed in the indirect tensile specimen, fatigue life should be evaluated in terms of stress difference, which for the indirect tensile test is equal to $4\sigma_T$. Expressing fatigue life in terms of stress difference changes the value of K_2 but does not affect the value of n_2 so that the relationship can be expressed in the form of Eq 2.6:

$$N_f = K_2' \left(\frac{1}{\Delta\sigma}\right)^{n_2}$$

where

TABLE 9. VALUES OF K_2, K_2', AND n_2 FROM THE LEAST SQUARES ANALYSIS:

$$N_f = K_2\left(\frac{1}{\sigma_T}\right)^{n_2} = K_2'\left(\frac{1}{\Delta\sigma}\right)^{n_2}$$

Temperature, °F	Aggregate	Asphalt Content, %	Number of Specimens	K_2	K_2'	n_2	Correlation Coefficient R*	Standard Error of Estimate S_e*
Individual Mixtures and Temperatures								
50	Limestone	6	9	4.59×10^{10}	8.20×10^{12}	3.74	0.99	0.06
	Gravel	6	9	1.90×10^{13}	2.53×10^{16}	5.19	0.99	0.06
75	Limestone	4	15	9.36×10^{5}	3.81×10^{7}	2.67	0.95	0.14
		5	20	8.75×10^{6}	4.63×10^{8}	2.86	0.98	0.09
		6	27	5.63×10^{7}	4.71×10^{9}	3.19	0.98	0.09
		7	36	4.29×10^{8}	8.75×10^{10}	3.84	0.99	0.14
		8	14	4.20×10^{7}	4.26×10^{9}	3.33	0.99	0.07
	Gravel	4	14	1.62×10^{6}	1.13×10^{8}	3.06	0.97	0.11
		5	21	5.43×10^{6}	3.30×10^{8}	2.96	0.97	0.10
		6	28	1.09×10^{8}	1.33×10^{10}	3.46	0.97	0.13
		7	40	8.68×10^{8}	2.60×10^{11}	4.11	0.96	0.27
		8	14	1.86×10^{7}	1.41×10^{9}	3.13	0.91	0.22
100	Limestone	6	9	5.27×10^{5}	2.09×10^{7}	2.66	1.00	0.06
	Gravel	6	9	3.26×10^{5}	1.41×10^{7}	2.72	0.99	0.10
Combined Mixtures and Temperatures								
50	LS & GR	6	18	9.35×10^{11}	4.56×10^{14}	4.46	0.94	0.16
75	Limestone	4,5,6,7,& 8	112	1.19×10^{8}	1.74×10^{10}	3.60	0.88	0.37
	Gravel	4,5,6,7,& 8	117	2.35×10^{8}	5.28×10^{10}	3.91	0.83	0.47
	LS & GR	4,5,6,7,& 8	229	1.56×10^{8}	8.40×10^{11}	3.73	0.85	0.43
100	LS & GR	6	18	4.14×10^{5}	1.71×10^{7}	2.69	0.96	0.17
50, 75,& 100	Limestone	4,5,6,7,& 8	130	2.76×10^{5}	2.94×10^{6}	1.71	0.56	0.62
	Gravel	4,5,6,7,& 8	135	3.00×10^{5}	3.77×10^{6}	1.83	0.52	0.69
	LS & GR	4,5,6,7,& 8	265	2.79×10^{5}	3.20×10^{6}	1.76	0.54	0.66

*For the general equation $\log N_f = \log K_2 - n_2 \log \sigma_T$

or $\log N_f = \log K_2' - n_2 \log \Delta\sigma$

σ_T = applied tensile stress

$\Delta\sigma$ = stress difference $= 4\sigma_T$

$\Delta\sigma$ = stress difference, or algebraic difference between horizontal and vertical stresses in the indirect tensile test,

K_2' = a constant, depending on mixture properties and testing temperature, and

N_f and n_2 are as previously defined.

The resulting relationships are shown in Figs 45 and 46 for limestone and gravel mixtures, respectively. The slopes n_2 in Figs 45 and 46 are the same as in Figs 43 and 44 for stress-fatigue life; the relationships, however, have been shifted to the right so that values of K_2' are larger than values of K_2. A high value of K_2 or K_2' indicates a relatively high value of fatigue life at low stresses; however, a high value of n_2 indicates a steep slope for the relationships (Figs 43 through 46).

Values of K_2' ranged from 1.41×10^7 to 2.53×10^{16} (Table 9). Even though larger, the values of K_2' still are generally smaller than those reported for other mixtures from flexural and axial load tests, which is compatible with the findings of Porter and Kennedy (Ref 63). These lower values are attributed to the higher testing temperature and the longer load duration (Ref 63) used in this testing program. However, both n_2 and K_2' compare well with those obtained by Navarro and Kennedy (Ref 55 and Appendix A) for inservice blackbase and asphalt concrete.

<u>Relationships between n_2 and K_2 Values</u>. Approximate linear relationships in the form

$$n_2 = A_2 + B_2 \log K_2$$

and

$$n_2 = A_2' + B_2' \log K_2'$$

were found to exist between n_2 and the logarithm of K_2 and between n_2 and the logarithm of K_2'. The relationships developed from a regression analysis were

$$n_2 = 0.919 + 0.312 \log K_2, \ (R = 0.95,\ S_e = 0.23) \qquad (5.1)$$

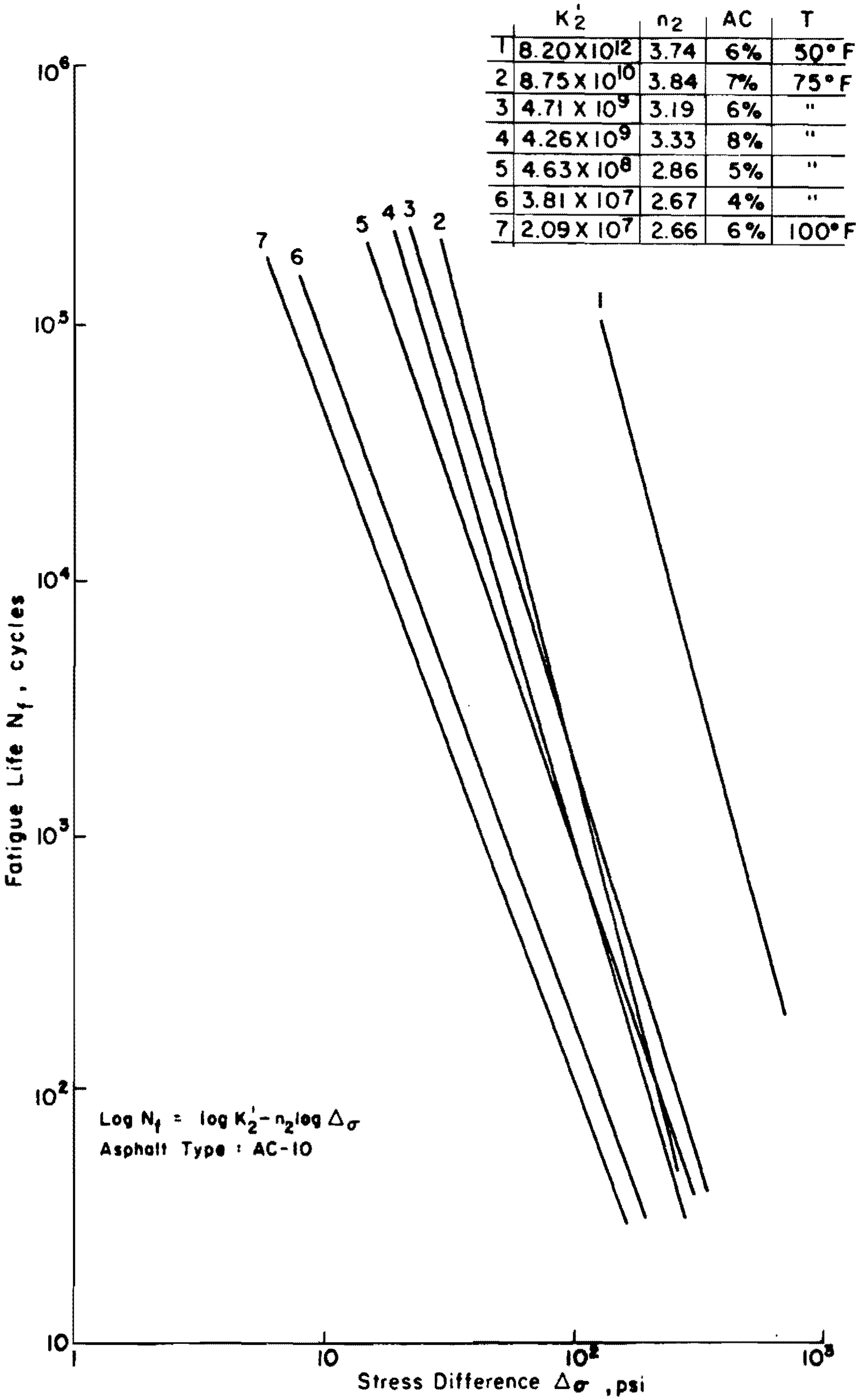


Fig 45. Relationships between fatigue life and stress difference for limestone mixtures.

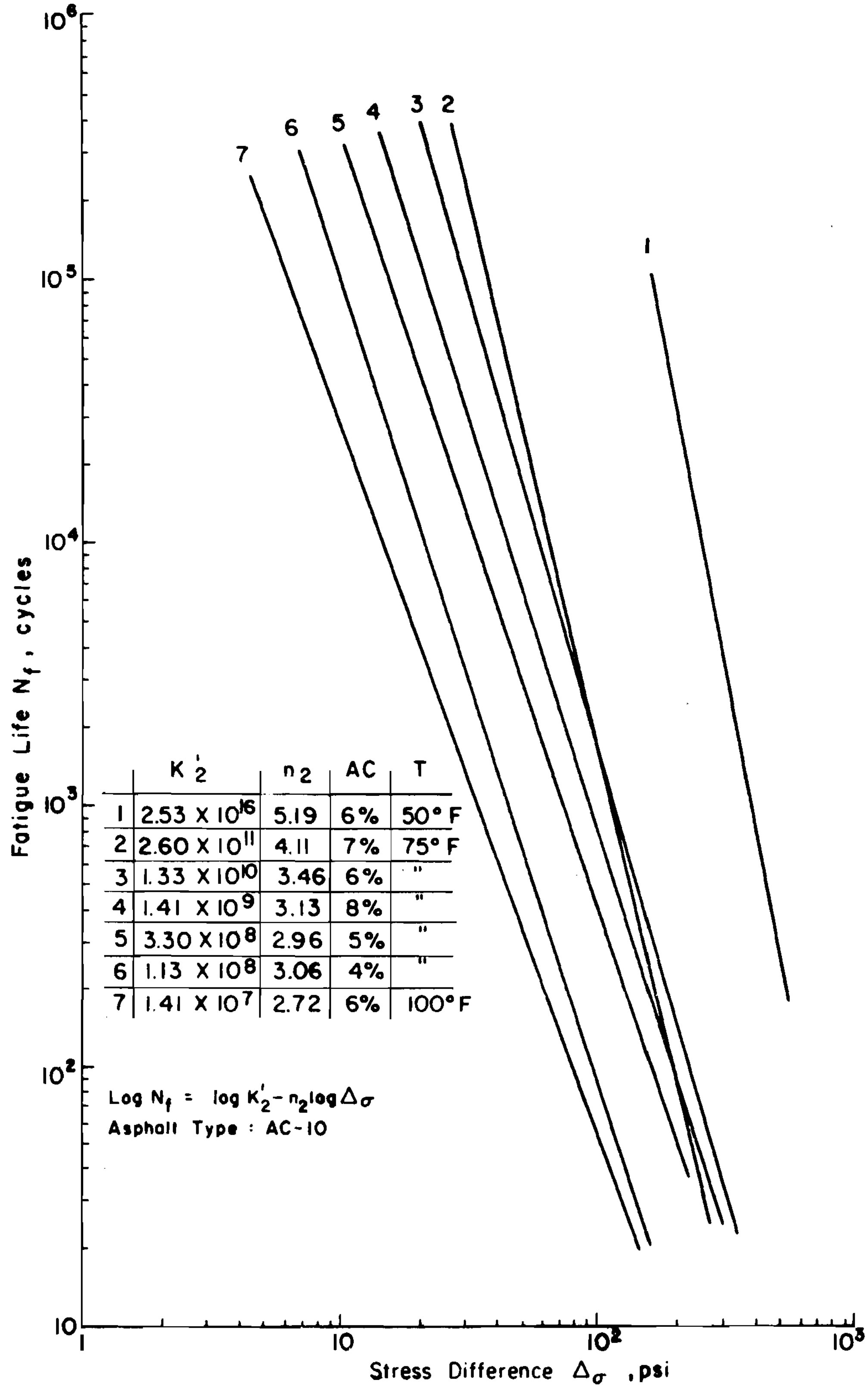


Fig 46. Relationships between fatigue life and stress difference for gravel mixtures.

and

$$n_2 = 0.734 + 0.266 \log K_2', \quad (R = 0.96,\ S_e = 0.19) \qquad (5.2)$$

The 95 percent confidence interval for R in Eq 5.1 was between 0.85 and 0.98 while that for R in Eq 5.2 was between 0.88 and 0.99. By making n_2 the independent variable rather than the dependent variable, the following relationships were obtained:

$$\log K_2 = 2.886\ n_2 - 1.867, \quad (R = 0.95,\ S_e = 0.70) \qquad (5.3)$$

and

$$\log K_2' = 3.485\ n_2 - 1.859, \quad (R = 0.96,\ S_e = 0.70) \qquad (5.4)$$

The high correlation coefficients associated with the above relationships make it possible to estimate one of the constants in terms of the other constant and thus possibly could reduce the number of constants in the relationship between fatigue life and stress (Eq 2.5) or fatigue life and stress difference (Eq 2.6).

Figure 47 illustrates the relationships of these prediction equations relative to the actual values of n_2, K_2, and K_2'.

A regression analysis was conducted to establish the relationship between K_2 and n_2 values reported by other investigators (Table 10) in which various mixtures and test procedures were used (Refs 32, 48, 55, and 60 and Appendix A) and it was found that linear relationships also existed between n_2 and $\log K_2$. The regression lines for individual studies are presented in Fig 48 while the values of the constants of the regression analysis together with the correlation coefficients and standard errors of estimate are shown in Table 11. Due to the relatively high correlation coefficients obtained for the individual studies, a further analysis was conducted on the combined data from all studies. Values for the dynamic indirect tensile test

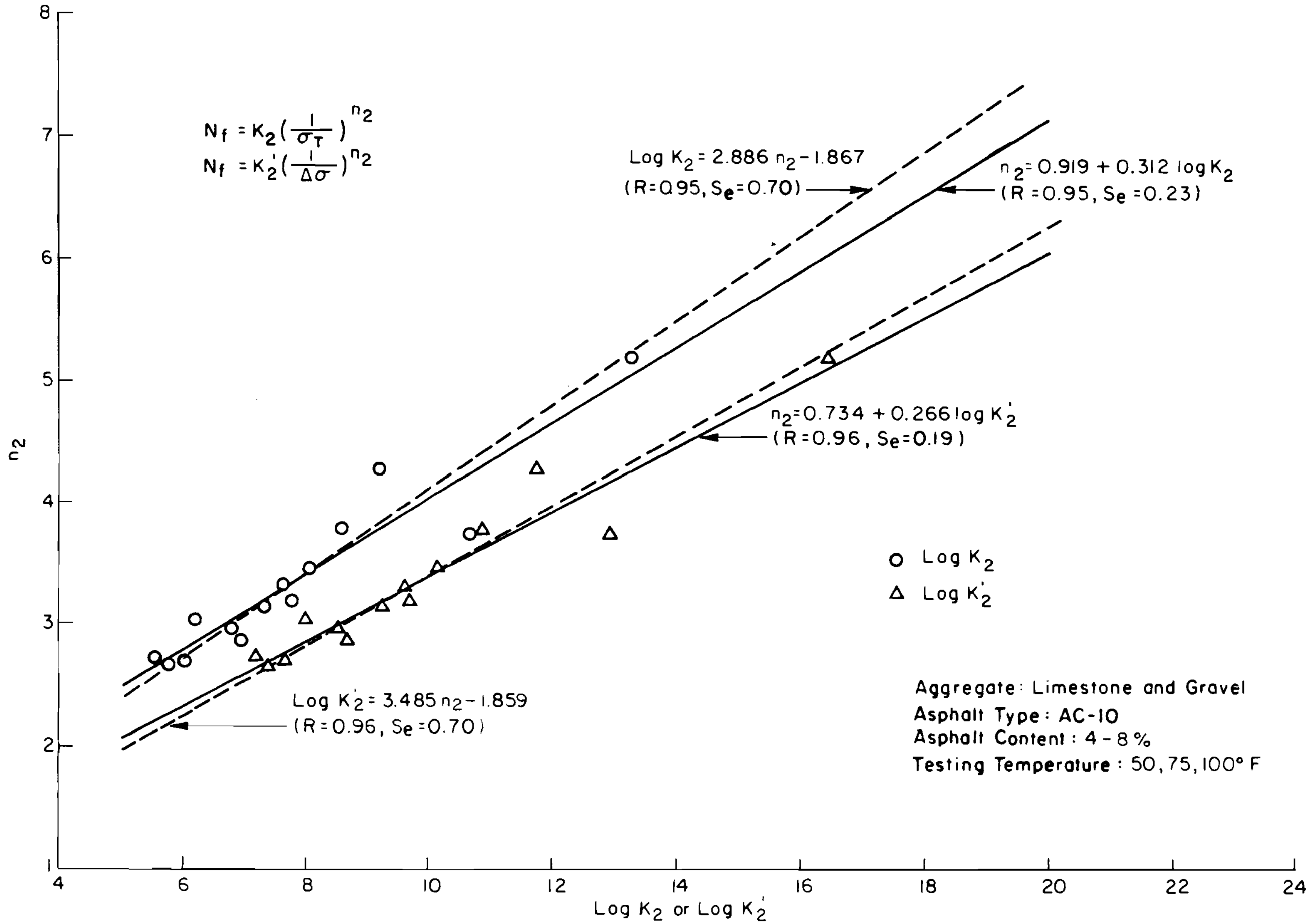


Fig 47. Relationships between n_2 and Log K_2 or Log K_2'.

TABLE 10. STUDIES FROM WHICH K_1, n_1, K_2, AND n_2 VALUES WERE OBTAINED

Test	Group	Source	Mix*
Flexure (Beam)	1	Monismith et al (Ref 48)	British, California, Gonzales, Morro Bay, and Folsom
	2	Kallas and Pazinauskas (Ref 32)	Colorado, Ontario, Washington State Test Track, and Laboratory Study asphalt concrete
Flexure (Rotating Cantilever)	3	Pell and Cooper (Ref 60)	Hot rolled asphalt (HRA), Dense bitumen macadam (DBM), Dense tar macadma (DTM), Asphalt concrete (AC), and Mastic asphalt (MA)
Axial Load	4		Hot rolled asphalt
Indirect Tension	5**	Navarro and Kennedy (Ref 55)	Inservice blackbase and asphalt concrete
	6 7**	This Study	Asphalt concrete or blackbase

*See references for details of mix and testing temperatures.

**K_2' values were used.

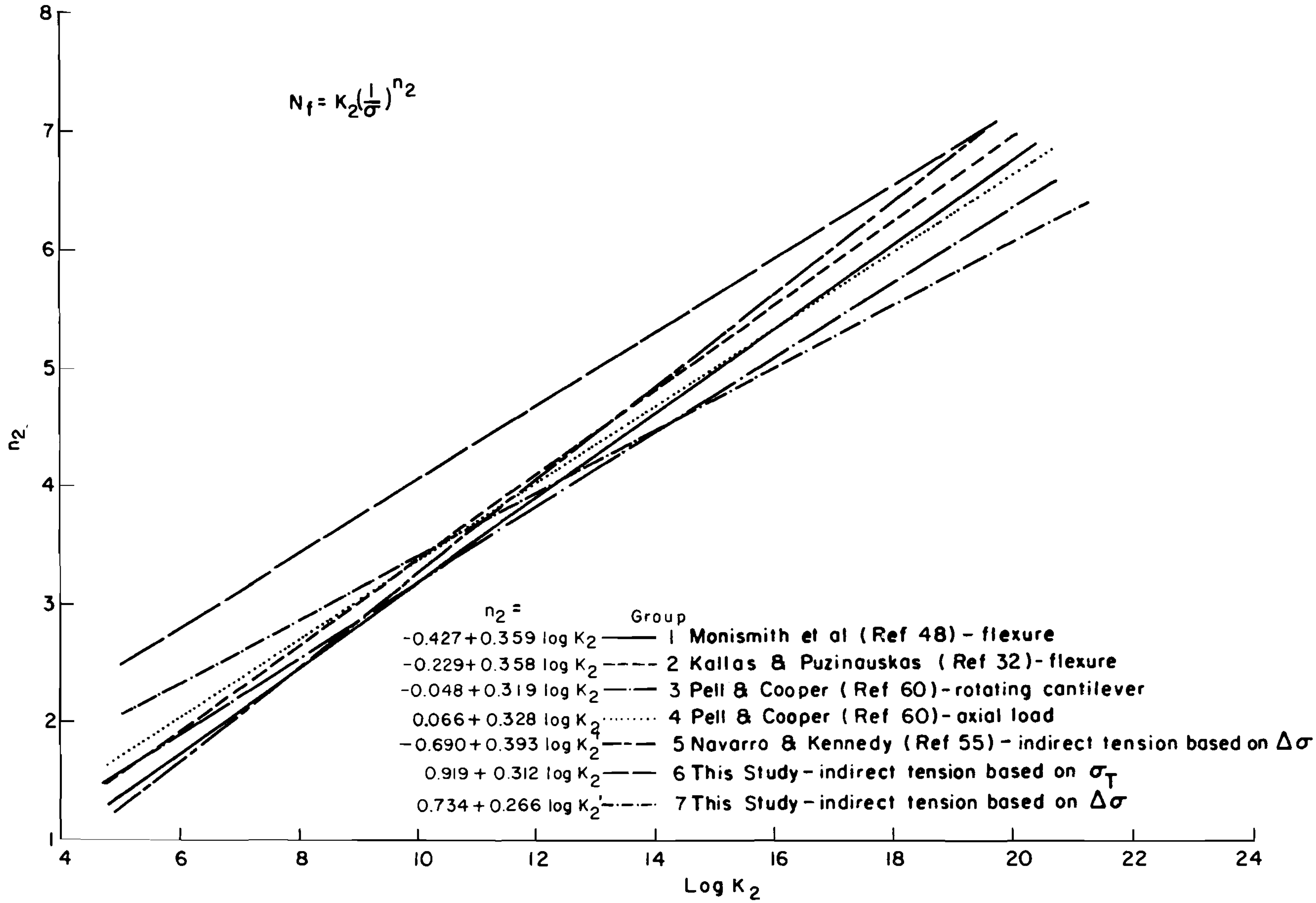


Fig 48. Relationships between n_2 and K_2 for various mixtures and test procedures.

TABLE 11. VALUES OF CONSTANTS FOR THE LEAST SQUARES REGRESSION EQUATIONS:

$$n_2 = A_2 + B_2 \log K_2 \quad \text{and} \quad \log K_2 = C_2 + D_2 n_2$$

(n_2 and K_2 are constants of the equation $N_f = K_2 \left(\frac{1}{\sigma}\right)^{n_2}$)

Group*	Number of Mixtures	A_2	B_2	Correlation Coefficient R	Standard Error of Estimate S_e	C_2	D_2	Correlation Coefficient R	Standard Error of Estimate S_e
1	19	-0.427	0.359	0.91	0.40	3.488	2.297	0.91	1.02
2	10	-0.229	0.358	0.91	0.35	3.251	2.330	0.91	0.90
3	38	-0.048	0.319	0.97	0.25	0.967	2.962	0.97	0.76
4	5	0.066	0.328	1.00	0.08	-0.114	3.028	1.00	0.23
5	13	-0.690	0.393	0.98	0.26	2.200	2.425	0.98	0.65
6	14	0.919	0.312	0.95	0.23	-1.867	2.886	0.95	0.70
7	14	0.734	0.266	0.96	0.19	-1.859	3.485	0.96	0.70
1,2,3,4,5, & 7	99	0.069	0.322	0.96	0.32	0.860	2.869	0.96	0.97

*See Table 10

were taken to be n_2 and K_2'. This analysis yielded the following average relationships between n_2 and $\log K_2$:

(1) with n_2 as dependent variable,

$$n_2 = 0.069 + 0.322 \log K_2, \quad (R = 0.96, S_e = 0.32) \qquad (5.5)$$

(2) with n_2 as independent variable,

$$\log K_2 = 0.860 + 2.869\, n_2, \quad (R = 0.96, S_e = 0.97) \qquad (5.6)$$

The 95 percent confidence interval for R was between 0.94 and 0.97. Figure 49 shows the positions of the above relationships relative to all data points.

Because of the high correlation coefficient obtained for the combination of all groups of studies, it was felt that a relationship probably exists between K_2 and n_2, irrespective of mixture properties or test procedures.

<u>Relationships with Initial Strain</u>

Fatigue life relationships are often expressed in terms of initial strain for controlled-stress tests and repeated strain for controlled-strain tests. From test results the initial strains were estimated in three different ways, (1) by projecting the relationship between resilient strain and the number of load applications to the first load application, (2) by dividing the applied dynamic stress by the average repeated-load resilient modulus, and (3) by dividing the applied dynamic stress by the average static modulus of elasticity. These estimated strain values were used to develop relationships between the logarithm of strain and the logarithm of fatigue life. From the results obtained, it was felt that the third method of estimating strain was better than the other two methods since it produced the highest correlation coefficients. The third method is also quicker and easier than the other two methods because it does not involve expensive and time consuming repeated-load tests. Therefore, further discussions on the relationships between fatigue life and initial strain are based on strains estimated as the ratio of applied dynamic stress to the average static modulus of elasticity.

The resulting relationships for the various mixtures containing limestone and gravel aggregates are shown in Figs 50 and 51 respectively. A regression

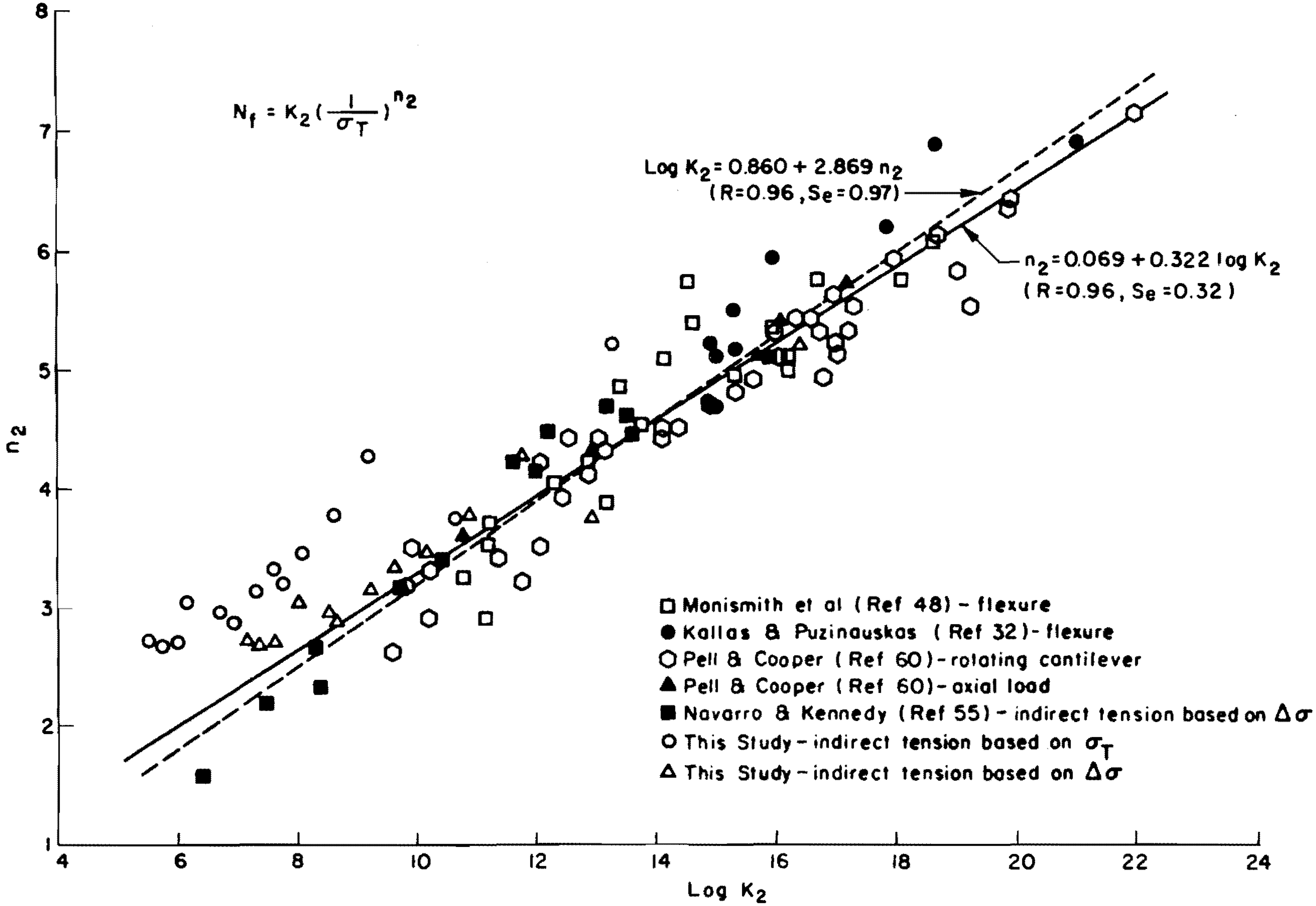


Fig 49. Relationships between n_2 and K_2 from various studies.

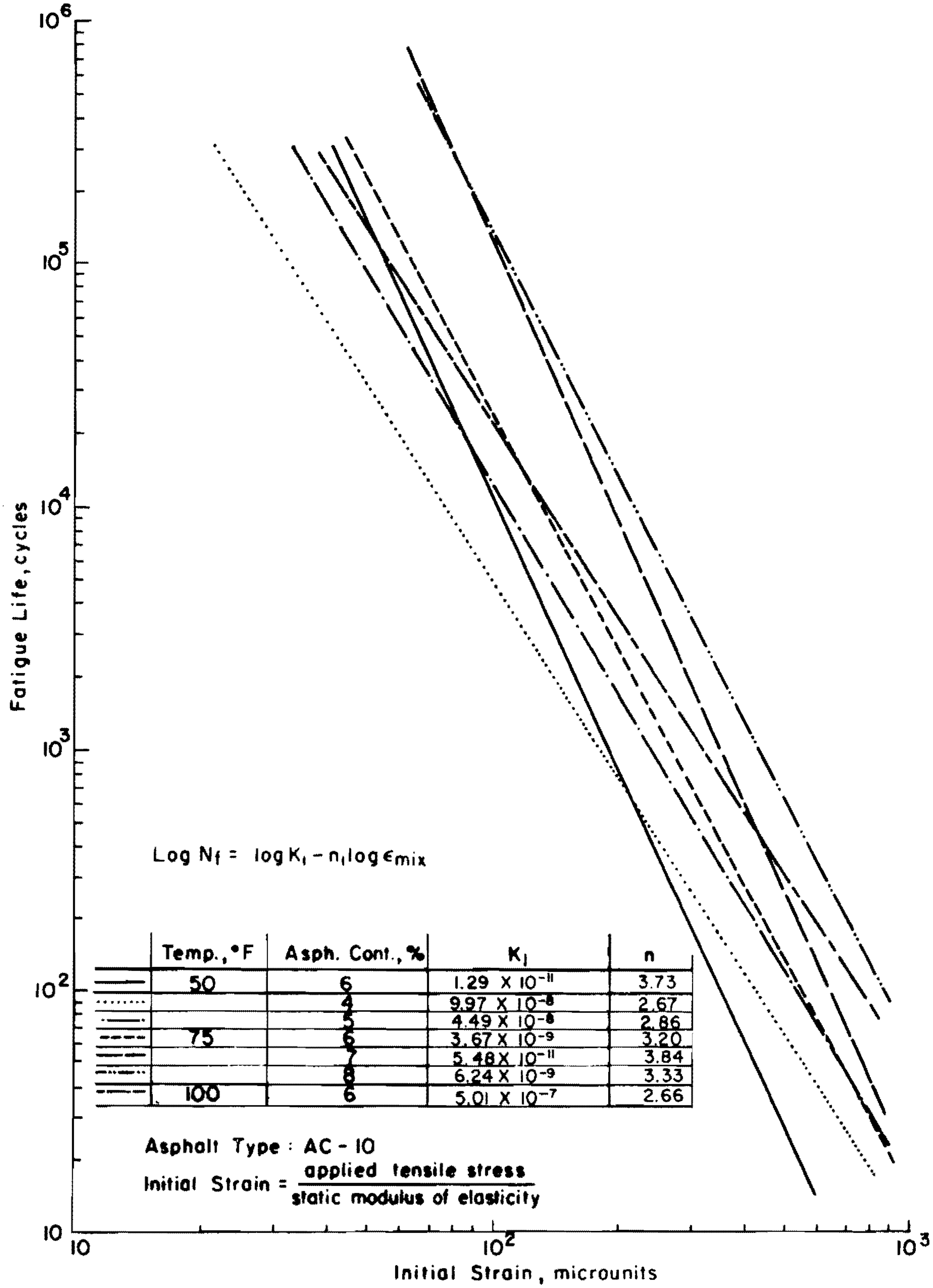


Fig 50. Relationships between fatigue life and initial strain for limestone mixtures.

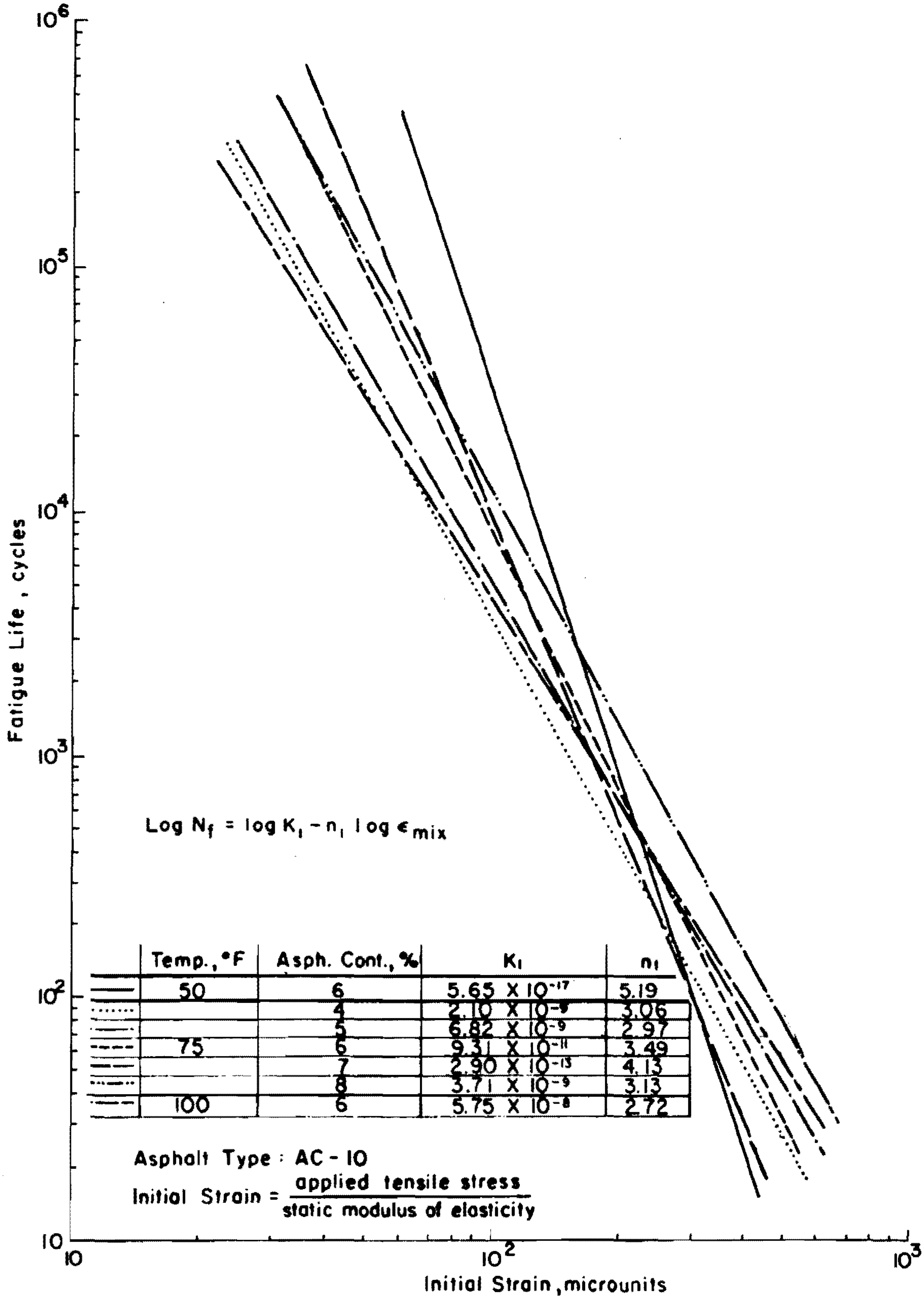


Fig 51. Relationship between fatigue life and initial strain for gravel mixtures.

analysis was used to establish these relationships and to obtain values of the constants K_1 and n_1 for the general equation (Eq 2.7)

$$N_f = K_1 \left(\frac{1}{\varepsilon_{mix}} \right)^{n_1}$$

where

N_f = fatigue life,
ε_{mix} = initial strain in the mixture, and
K_1 and n_1 = constants.

Values of K_1 and n_1 are summarized in Table 12 and are indicated in Figs 50 and 51, which show that a low value of K_1 indicates a relatively high value of fatigue life. However, a high value of n_1 indicates a steep slope in the strain-fatigue life relationships. Values of K_1 ranged from 5.65×10^{-17} to 5.01×10^{-7} and of n_1 ranged from 2.66 to 5.19. These values were compared to those obtained from previous flexural and axial load tests on other mixtures and are summarized in Appendix A. This summary indicates that K_1 generally ranges from 5.0×10^{-20} to 5.0×10^{-5} while n_1 ranges from about 2.5 to 6.3, depending on asphalt content, testing temperature, and testing procedure. The values of K_1 and n_1 obtained using the dynamic indirect tensile test compare favorably with previously reported values, and values from this study were essentially the same as those obtained for similar mixtures with the same asphalt contents and tested at the same temperature.

Thus, it appears that comparable values of K_1 and n_1 can be obtained for essentially the same mixtures, irrespective of test procedures. Since the mechanism of failure is probably strain dependent, consideration of strain at least partially accounts for the states of stress, temperature, and load pulse.

<u>Relationship between K_1 and n_1 Values</u>. A linear relationship was also found to exist between n_1 and $\log K_1$ (Fig 52), as follows:

(a) n_1 as dependent variable

$$n_1 = 0.938 - 0.261 \log K_1, \quad (R = 0.98,\ S_e = 0.13) \qquad (5.7)$$

TABLE 12. VALUES OF CONSTANTS FOR THE LEAST SQUARES REGRESSION EQUATION: $N_f = K_1 \left(\frac{1}{\varepsilon_{mix}}\right)^{n_1}$

(ε_{mix} = initial mixture strain, ratio of repeated stress to static modulus of elasticity)

Temperature, °F	Aggregate*	Asphalt Content, %	Number of Specimens	K_1	n_1	Correlation Coefficient R	Standard Error of Estimate S_e
Individual Mixtures and Temperatures							
50	Limestone	6	9	1.29×10^{-11}	3.73	0.99	0.06
	Gravel	6	9	5.65×10^{-17}	5.19	0.99	0.06
75	Limestone	4	15	9.97×10^{-8}	2.67	0.95	0.14
		5	20	4.49×10^{-8}	2.86	0.98	0.09
		6	27	3.67×10^{-9}	3.20	0.98	0.09
		7	36	5.48×10^{-11}	3.84	0.99	0.14
		8	14	6.24×10^{-9}	3.33	0.99	0.07
	Gravel	4	14	2.10×10^{-9}	3.06	0.97	0.11
		5	21	6.82×10^{-9}	2.97	0.97	0.11
		6	28	9.31×10^{-11}	3.49	0.97	0.13
		7	40	2.90×10^{-13}	4.13	0.96	0.27
		8	14	3.71×10^{-9}	3.13	0.91	0.22
100	Limestone	6	9	5.01×10^{-7}	2.66	1.00	0.06
	Gravel	6	9	5.75×10^{-8}	2.72	0.99	0.10
Combined Mixtures and Temperatures							
50	Ls & gr	6	18	2.96×10^{-13}	4.17	0.97	0.11
75	Limestone	4,5,6,7,& 8	112	4.58×10^{-9}	3.22	0.87	0.38
	Gravel	4,5,6,7,& 8	117	3.86×10^{-11}	3.59	0.96	0.24
	Ls & gr	4,5,6,7,& 8	229	5.52×10^{-8}	2.82	0.82	0.46
100	Ls & gr	6	18	2.76×10^{-5}	2.05	0.79	0.36
50,75 & 100	Limestone	4,5,6,7,& 8	130	4.80×10^{-8}	2.92	0.86	0.38
	Gravel	4,5,6,7,& 8	135	6.06×10^{-11}	3.54	0.96	0.23
	Ls & gr	4,5,6,7,& 8	265	9.38×10^{-8}	2.76	0.83	0.44

*Ls - limestone, gr - gravel.

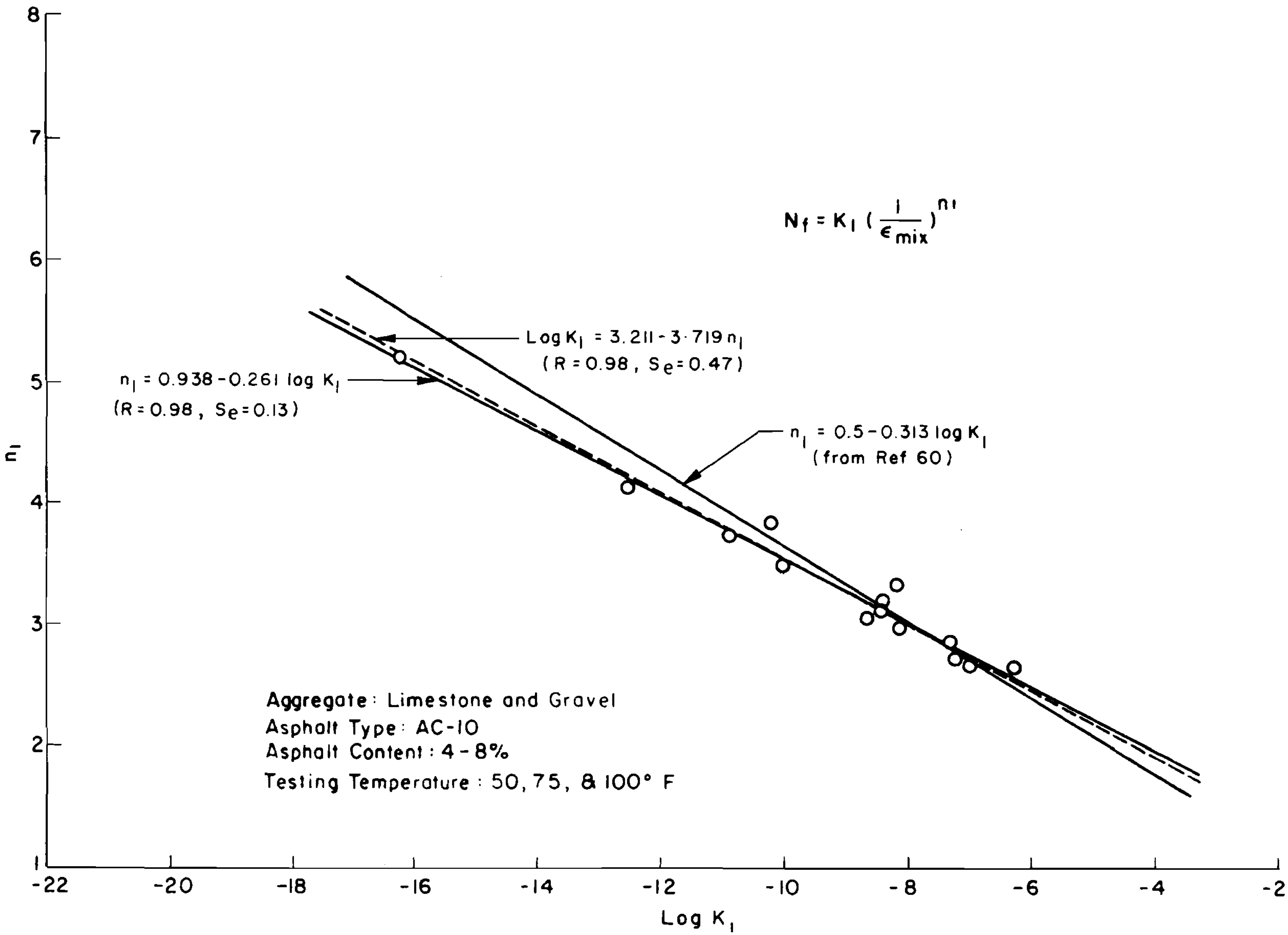


Fig 52. Relationships between n_1 and log K_1.

(b) $\log K_1$ as dependent variable

$$\log K_1 = 3.211 - 3.719\, n_1, \quad (R = 0.98,\ S_e = 0.47) \qquad (5.8)$$

The 95 percent confidence interval for R was between 0.94 and 0.99. Recently, Pell and Cooper (Ref 60) reported an expression (Eq 2.8) which was similar to Eq 5.7:

$$n_1 = 0.5 - 0.313 \log K_1$$

As shown in Fig 52, the equation obtained by Pell and Cooper fell within the data points obtained in this study using the dynamic indirect tensile test.

Because of this similarity, an evaluation of the relationship between n_1 and $\log K_1$ was also conducted for values obtained from various other test methods (Table 10) and for different kinds of mixtures (Refs 32, 48, 55, and 60, and Appendix A). Results of the regression analysis are shown in Table 13 and the relationships between n_1 and K_1 are illustrated in Fig 53 for each group of the study. By combining data from the various groups of studies, a regression analysis also indicated linear relationships as follows:

(1) n_1 as dependent variable

$$n_1 = 1.350 - 0.252 \log K_1, \quad (R = 0.95,\ S_e = 0.29) \qquad (5.9)$$

(2) n_1 as independent variable

$$\log K_1 = 3.977 - 3.609\, n_1, \quad (R = 0.95,\ S_e = 1.09) \qquad (5.10)$$

The 95 percent confidence interval for R was between 0.92 and 0.97. Figure 54 illustrates the positions of the above relationships relative to all data points.

The high correlation coefficients associated with the above expressions show that a linear relationship exists between n_1 and $\log K_1$, irrespective of mixture properties and test procedures.

<u>Focal Point of Strain-Fatigue Life Relationships</u>. Pell and Cooper obtained a focal point of intersection, i.e., a point at which all strain-fatigue life relationships intersect, which occurred at a strain of 6.3×10^{-4} and a fatigue life of 40.

TABLE 13. VALUES OF CONSTANTS FOR THE LEAST SQUARES REGRESSION EQUATIONS:

$n_1 = A_1 + B_1 \text{ Log } K_1$ and $\text{Log } K_1 = C_1 + D_1 n_1$

(n_1 and K_1 are constants of the equation $N_f = K_1 \left(\frac{1}{\epsilon_{mix}}\right)^{n_1}$)

Group	Number of Mixtures	A_1	B_1	Correlation Coefficient R	Standard Error of Estimate S_e	C_1	D_1	Correlation Coefficient R	Standard Error of Estimate S_e
1	19	1.443	-0.265	0.97	0.18	4.628	-3.537	0.97	0.65
2	10	1.454	-0.288	0.95	0.20	3.961	-3.149	0.95	0.65
3	38	0.782	-0.294	0.98	0.21	2.038	-3.256	0.98	0.70
4	5	1.373	-0.238	1.00	0.05	5.724	-4.193	1.00	0.20
5	—	—	—	—	—	—	—	—	—
6	14	0.938	-0.261	0.98	0.13	3.211	-3.719	0.98	0.47
1,2,3,4,& 6	86	1.350	-0.252	0.95	0.29	3.977	-3.609	0.95	1.09

*See Table 10

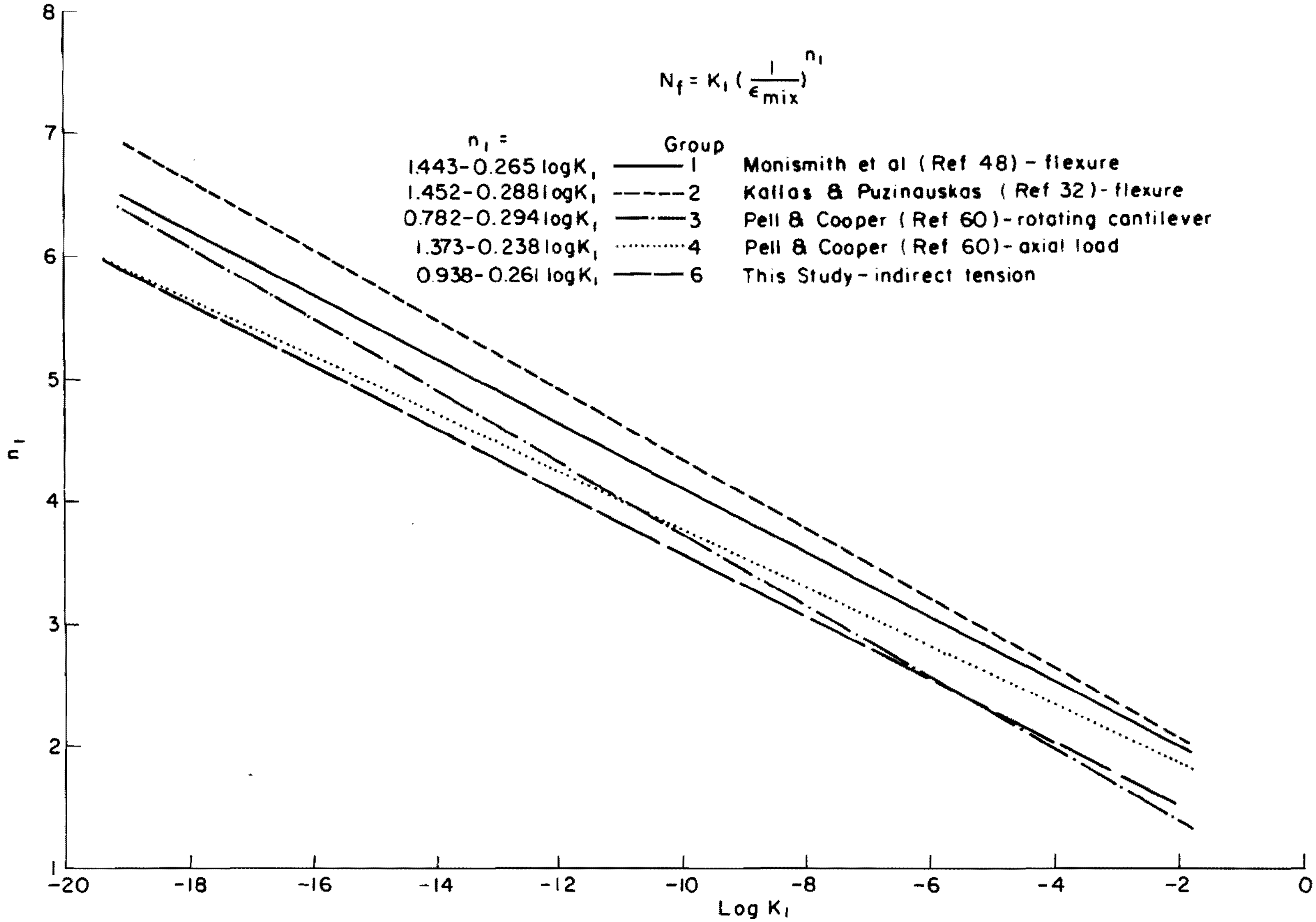


Fig 53. Individual relationships between n_1 and Log K_1 for various mixtures and test procedures.

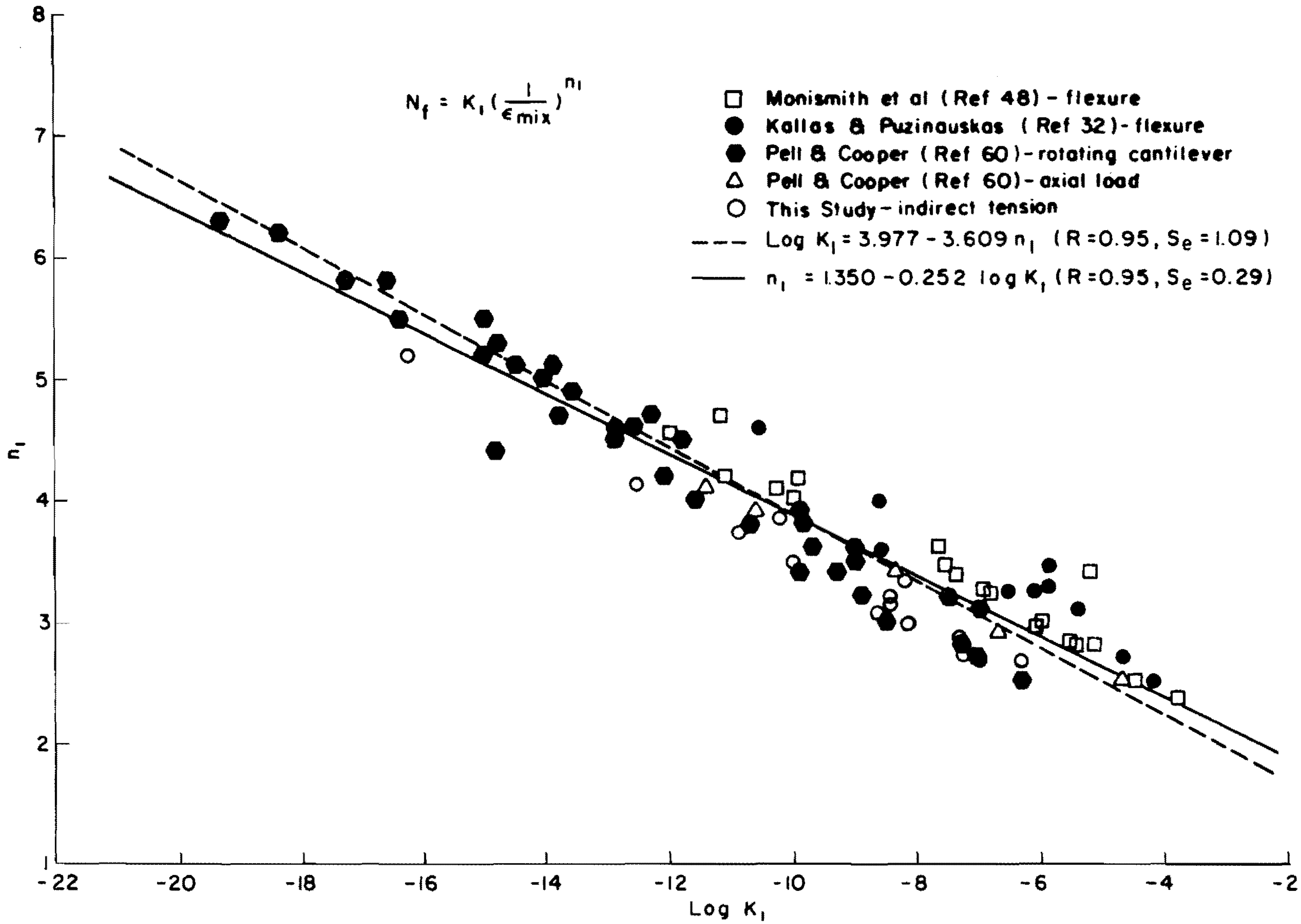


Fig 54. Combined relationships between n_1 and K_1 from various studies.

In this study, different focal points occurred and were found to depend on whether Eq 5.7 or 5.8 was used, i.e., whether n_1 or $\log K_1$ was made the dependent variable in the regression analysis. Using Eq 5.7, the focal point occurred at a strain of 1.47×10^{-4} and a fatigue life of 3,926. On the other hand, the use of Eq 5.8 resulted in a focal point at a strain of 1.91×10^{-4} and a fatigue life of 1,626. However, neither of these theoretical focal points was indicated in either Fig 50 or 51, which illustrate the relationships between initial strain and fatigue life. It is important, though, that a relationship exists between K_1 and n_1, which simplifies the fatigue life prediction based on its relationship with initial strain.

FACTORS AFFECTING FATIGUE LIFE

Previous studies have shown that the fatigue life of an asphalt mixture is dependent on many mixture and testing variables. In this study, the effects of asphalt content, aggregate type, and testing temperature were evaluated.

Asphalt Content

The relationships between fatigue life and asphalt content are shown in Fig 55 for the limestone and gravel mixtures tested at 75°F. The geometric means rather than the simple arithmetic means were used to obtain each point since it has been shown that fatigue life has a logarithmic normal distribution.

It is evident that there was an optimum asphalt content for maximum fatigue life. For the mixtures and stress levels used in this study, stress level had a negligible effect on the optimum asphalt content; however, for the limestone mixtures the optimum appeared to decrease slightly with increased stress level.

The optimum asphalt content for maximum fatigue life was generally in the range of 6 to 6.5 percent, which was essentially equal to the optimum for maximum strength and maximum static modulus of elasticity but was slightly less than the optimum asphalt content for maximum bulk density.

The effect of asphalt content on the regression coefficients K_1 and n_1 are shown in Fig 56, where it is seen that the maximum values of n_1 and the minimum values of K_1 occurred at an asphalt content of about 7 percent, which is slightly higher than the optimum asphalt content for maximum fatigue life. In addition, Fig 57 illustrates the effect of asphalt content on K_2,

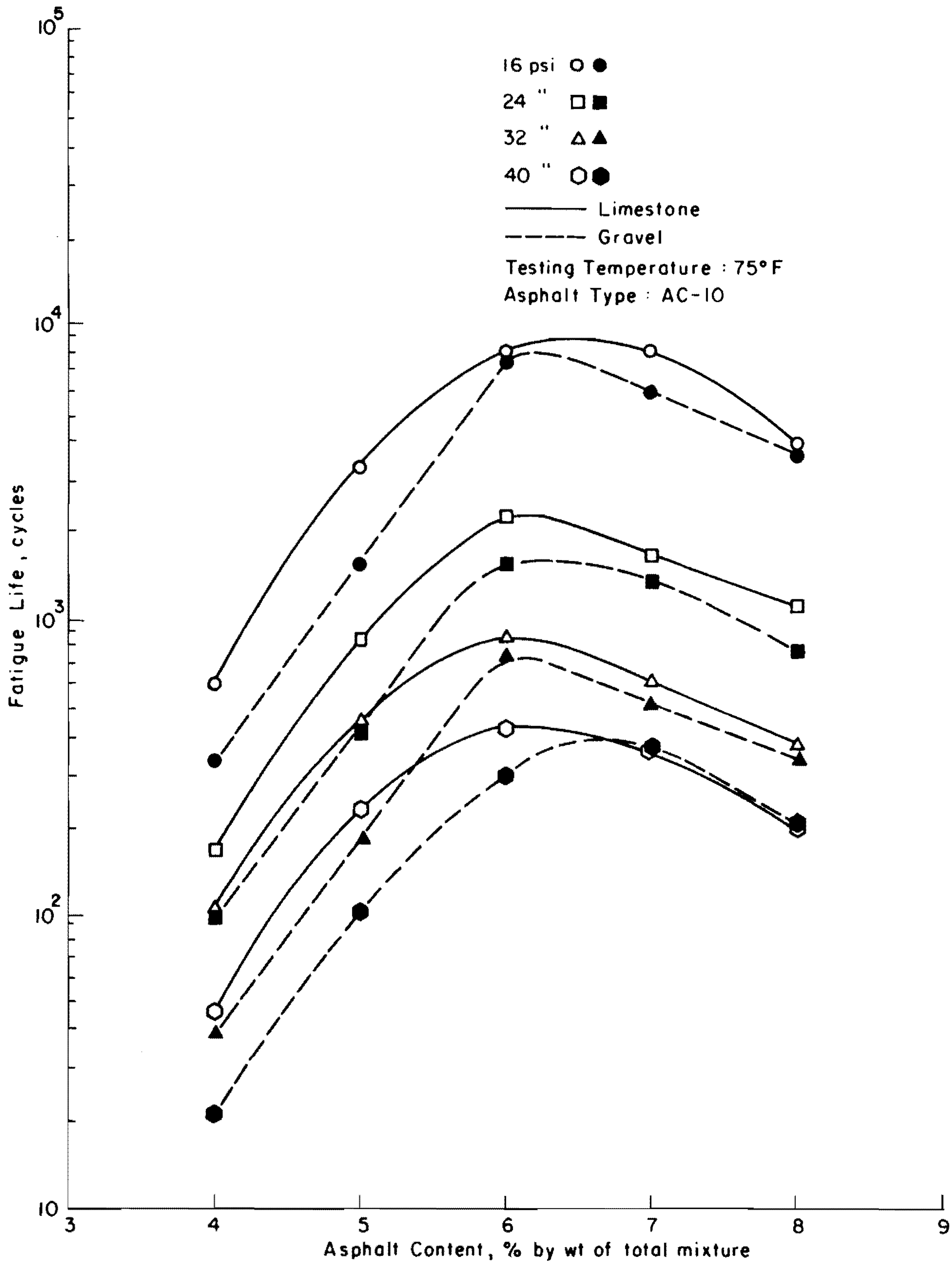


Fig 55. Effects of aggregate and asphalt content on fatigue life.

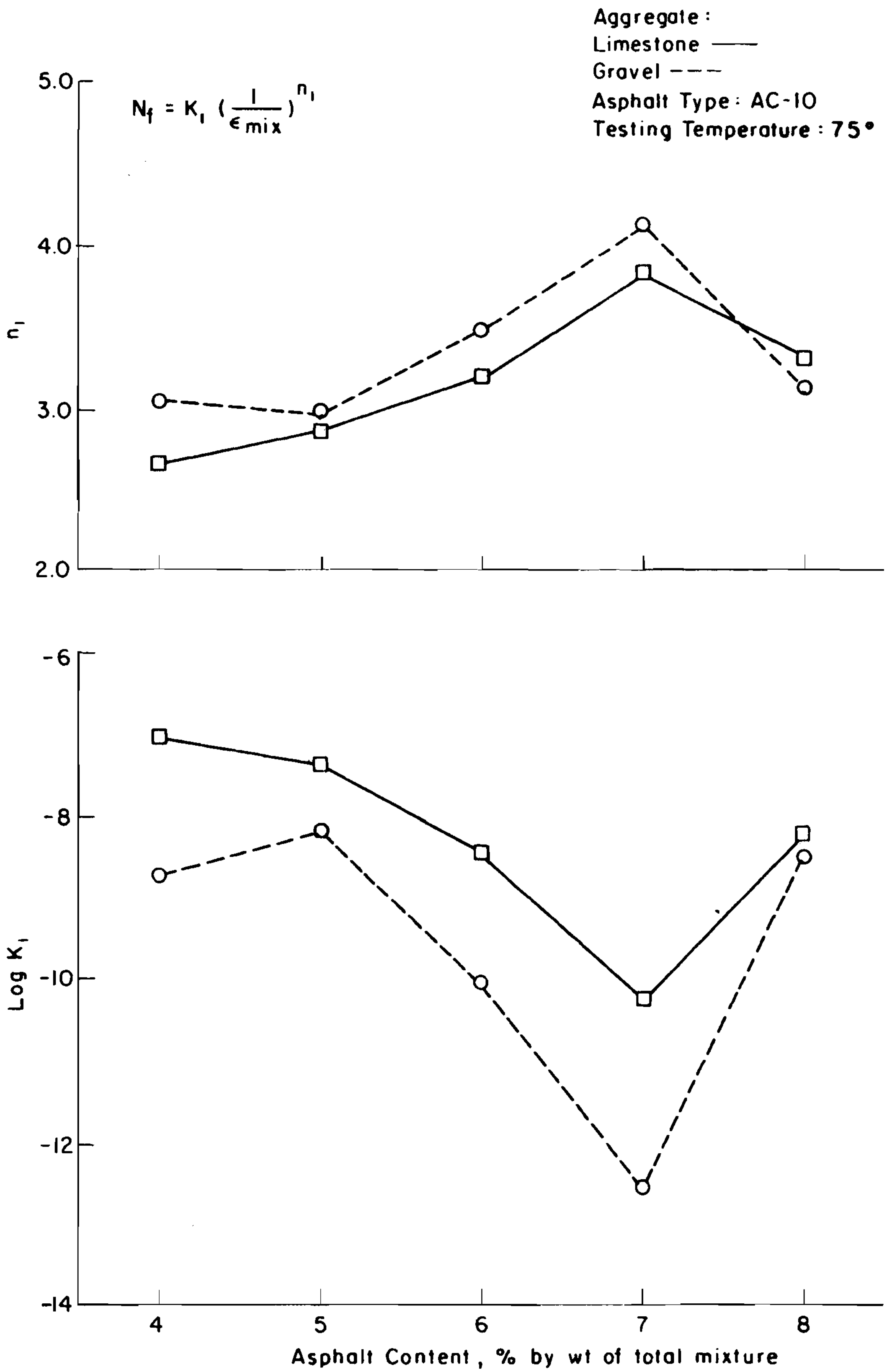


Fig 56. Effects of asphalt content on the values of K_1 and n_1 .

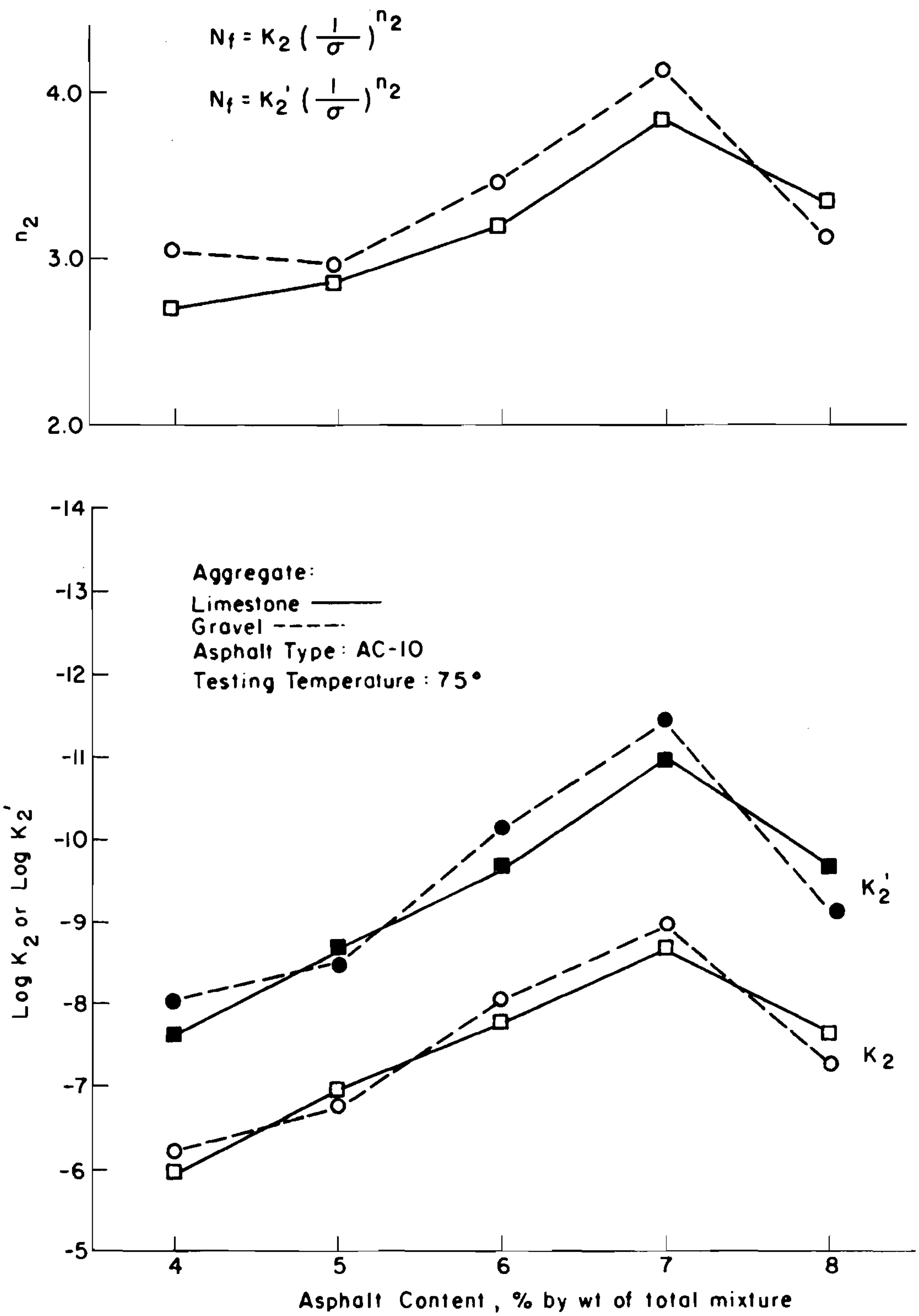


Fig 57. Effects of asphalt content on the values of K_2 and n_2.

K_2', and n_2. Maximum values of K_2, K_2', and n_2 occurred at an asphalt content of approximately 7 percent, also higher than the approximate optimum asphalt content for maximum fatigue life.

Aggregate Type

The effect of aggregate type for mixtures containing 6 percent asphalt and tested at 50, 75, and 100°F is shown in Fig 58. This figure indicates that there was a general tendency for the limestone mixtures to have slightly longer fatigue lives than the gravel mixtures. The same tendency was also observed for mixtures containing 4, 5, 6, and 8 percent asphalt and tested at 75°F (Table 8 and Fig 55).

It was also observed that the difference decreased with decreased stress and increased asphalt content and that, in some cases, the gravel mixtures had a longer fatigue life. In nearly all cases, however, the differences in fatigue lives for limestone and gravel mixtures were very small and it can be said that both mixtures exhibited approximately the same fatigue lives. In an earlier study on the same mixtures, Moore and Kennedy (Ref 52) reported that the gravel mixtures had longer fatigue lives than the limestone mixtures; however, this effect was not a significant behavioral difference. A significant interaction effect, however, was detected between aggregate and stress with the gravel exhibiting significantly longer fatigue lives at low stress levels.

Thus, it would appear that for the conditions of this study, aggregate was not important, but that for low stress levels such as those experienced in the field, aggregate type could be a significant variable.

From Figs 56 and 59, it can be seen that the gravel mixtures exhibited higher values of n_1 and lower values of K_1 than the limestone mixtures. In terms of the stress relationships, the gravel mixtures exhibited higher values of n_2, K_2 and K_2' than the limestone mixtures (Figs 57 and 60).

Temperature

The effect of temperature is illustrated in Figs 43, 44, and 58 and was found to agree with the findings of previous studies using stress-controlled tests. In general, fatigue life increased with decreasing temperatures for the three temperatures considered (50, 75, and 100°F).

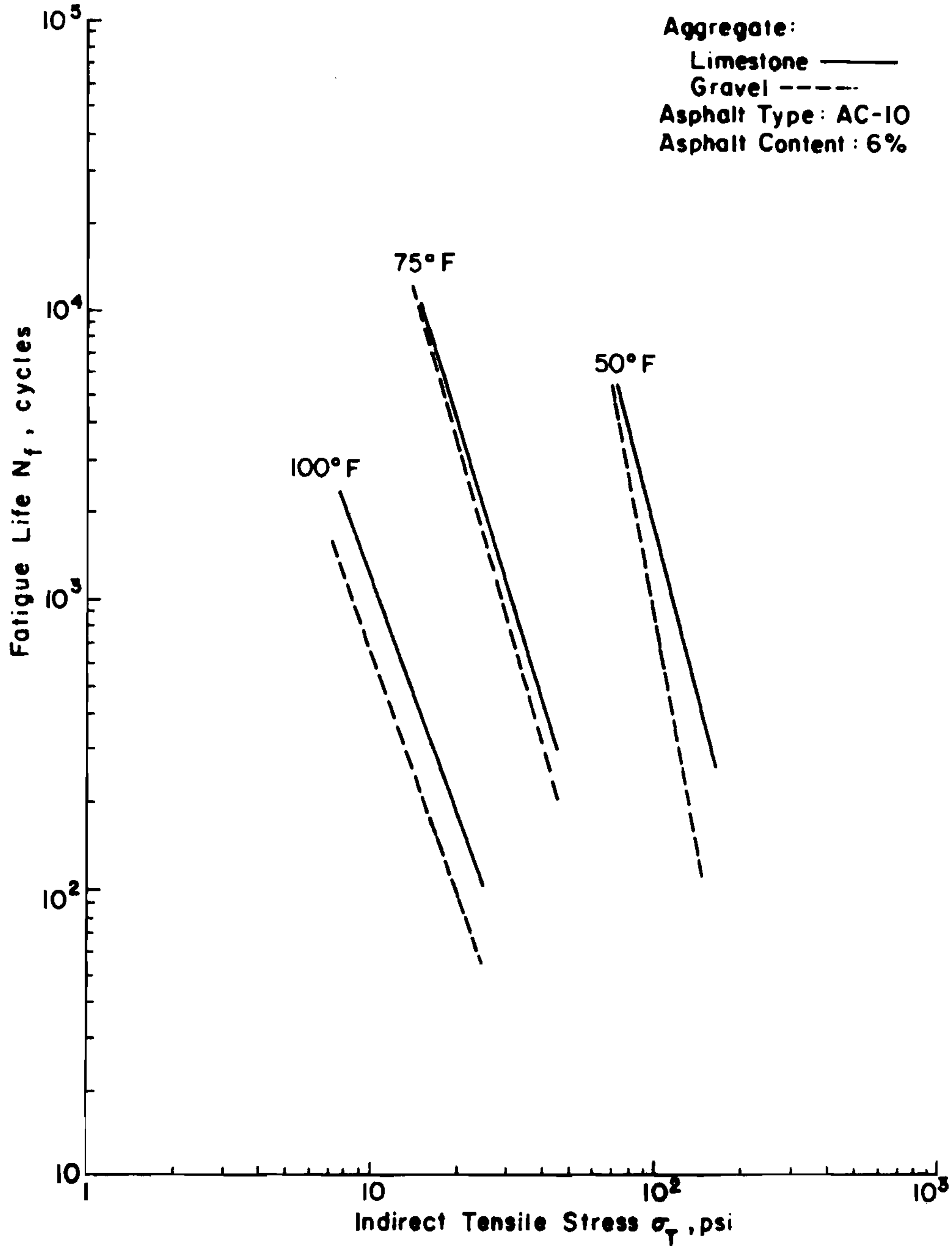


Fig 58. Effects of aggregate type and testing temperature on fatigue life.

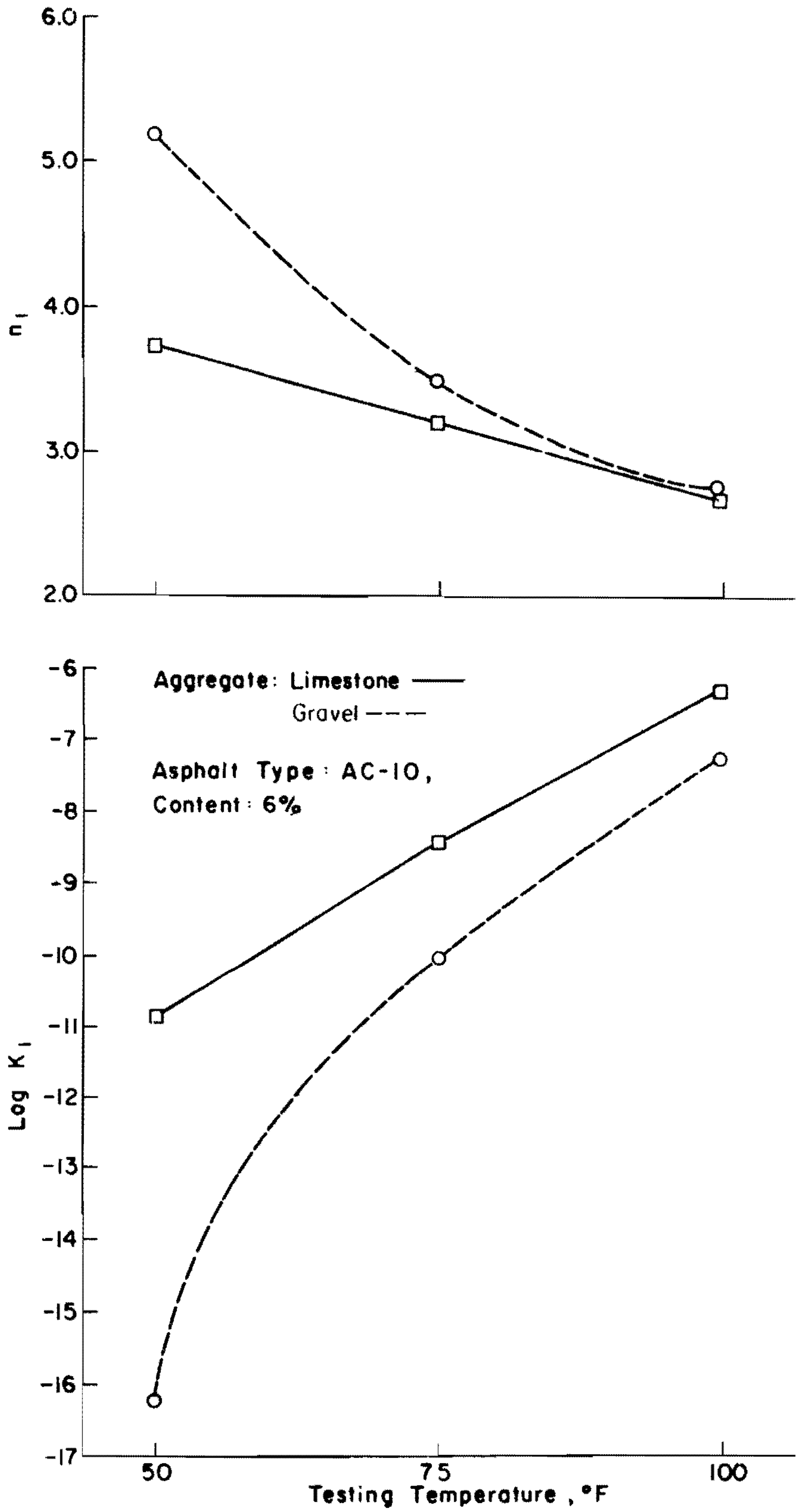


Fig 59. Effects of testing temperature on the values of K_1 and n_1.

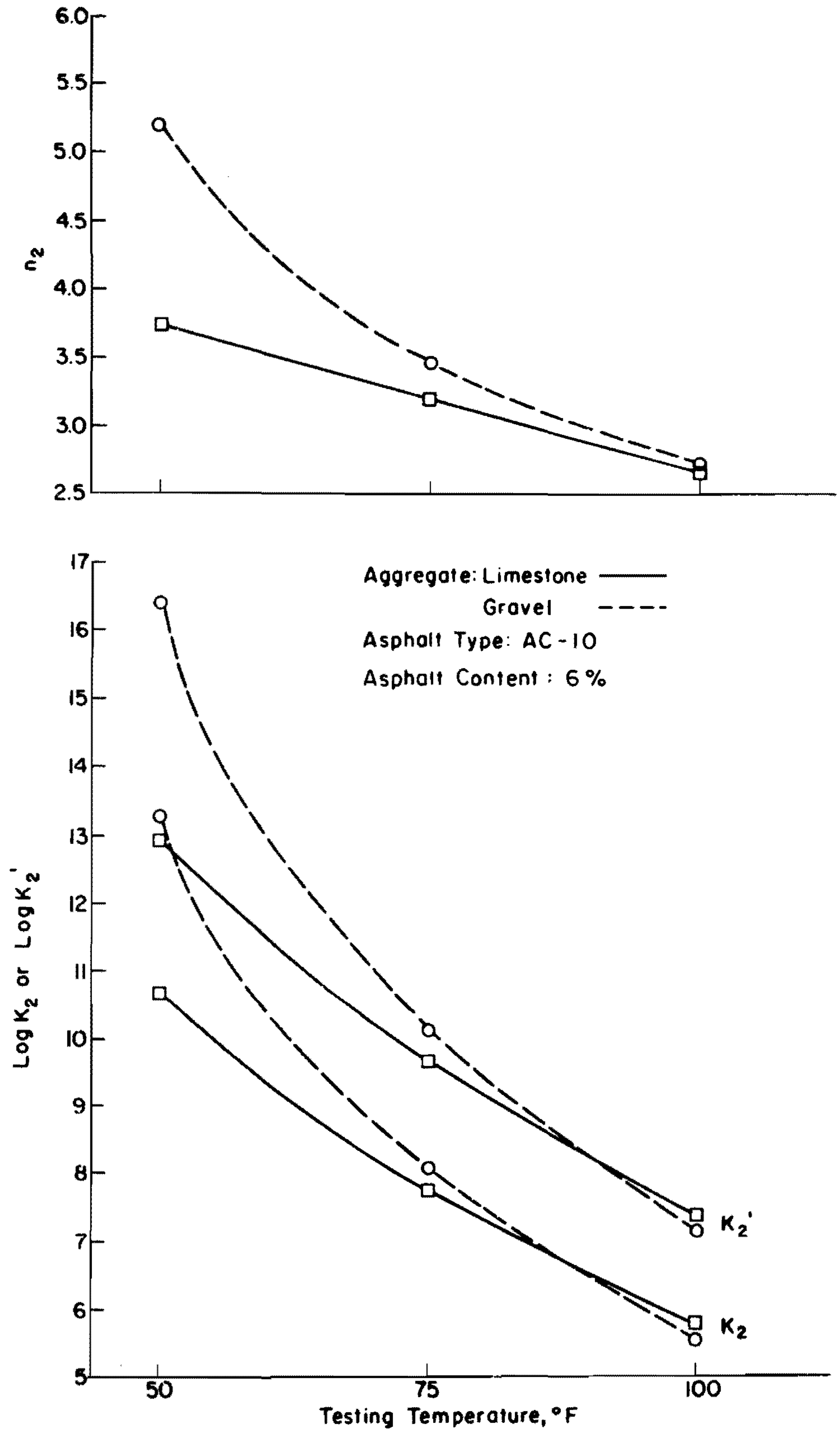


Fig 60. Effects of testing temperature on the values of K_2, K_2', and n_2.

The effect of temperature on the regression coefficients K_1 and n_1 is shown in Fig 59. An increase in testing temperature produced an increase in K_1 but a decrease in n_1. Figure 60 illustrates the effect of temperature on the regression coefficients K_2, K_2', and n_2; an increase in temperature produced a decrease in both K_2 and K_2' and also in n_2.

FATIGUE LIFE PREDICTION

Efforts were made to develop mathematical relationships to predict fatigue life on the basis of load variables or mixture properties in order to eliminate the need to conduct time consuming and costly repeated-load fatigue tests.

Relationships with Strain and Stress

The relationships between fatigue life and both initial strain and applied stress have been discussed previously. Previous discussion, however, was primarily concerned with the nature of the relationships and the effect of stress and strain on fatigue life. Discussion in this section will be concerned with the use of these relationships for estimating fatigue life.

Initial Strain. Previous consideration of the relationships between fatigue life and initial strain indicated that fatigue life can be predicted in terms of initial strain, defined as the ratio of repeated dynamic stress to static modulus of elasticity. Values of K_1 and n_1 (Table 12) for each asphalt content, testing temperature, and aggregate type can be used with the general equation in predicting fatigue life. However, more important is the fact that the following equations can be used irrespective of asphalt content and testing temperature:

Limestone mixtures: $$N_f = 4.80 \times 10^{-8} \left(\frac{1}{\epsilon_{mix}}\right)^{2.92} \tag{5.11}$$

$$(R^2 = 0.74,\ S_e = 0.38)$$

Gravel mixtures: $$N_f = 6.06 \times 10^{-11} \left(\frac{1}{\epsilon_{mix}}\right)^{3.54} \tag{5.12}$$

$$(R^2 = 0.92,\ S_e = 0.23)$$

In addition, a single overall expression was developed for estimating fatigue life in terms of initial strain, irrespective of asphalt content, testing temperature, and aggregate type:

$$N_f = 9.38 \times 10^{-8}\left(\frac{1}{\epsilon_{mix}}\right)^{2.76}, \ (R^2 = 0.70, \ S_e = 0.44) \qquad (5.13)$$

This relationship (Eq 5.13) was obtained with a coefficient of determination of 0.70 and a standard error of estimate (a measure of the variation of the logarithm of fatigue life about the regression line) of 0.44. Figure 61, which shows the plot of Eq. 5.13 with respect to all data points, indicates that most of the points lie within the 80 percent confidence interval and nearly all lie within the 95 percent interval.

The above expressions are considered to be relatively accurate; however, errors can be expected. In view of the inherent variable associated with fatigue behavior, it is felt that estimated fatigue lives would be relatively accurate.

Stress. The previously discussed relationships between applied stress and fatigue life can be used to estimate fatigue life. The values of K_2 and n_2 presented in Table 9 for each mixture and testing temperature can be used with the general stress-fatigue life equation to predict fatigue life.

As shown in Table 9, which contains the values of K_2 and n_2, the correlation coefficients are high for the relationships for each mixture. A further analysis was made by combining and analyzing all mixtures tested at a given temperature. The regression equations also show reasonably high correlation coefficients. Thus, within the range of stress levels and mixture properties utilized, approximate relationships for combined limestone and gravel mixtures can be expressed as follows:

50°F:

$$N_f = 4.56 \times 10^{14}\left(\frac{1}{\Delta\sigma}\right)^{4.46}$$

or

$$N_f = 9.35 \times 10^{11}\left(\frac{1}{\sigma_T}\right)^{4.46} \qquad (5.14)$$

$$(R^2 = 0.88, \ S_e = 0.16)$$

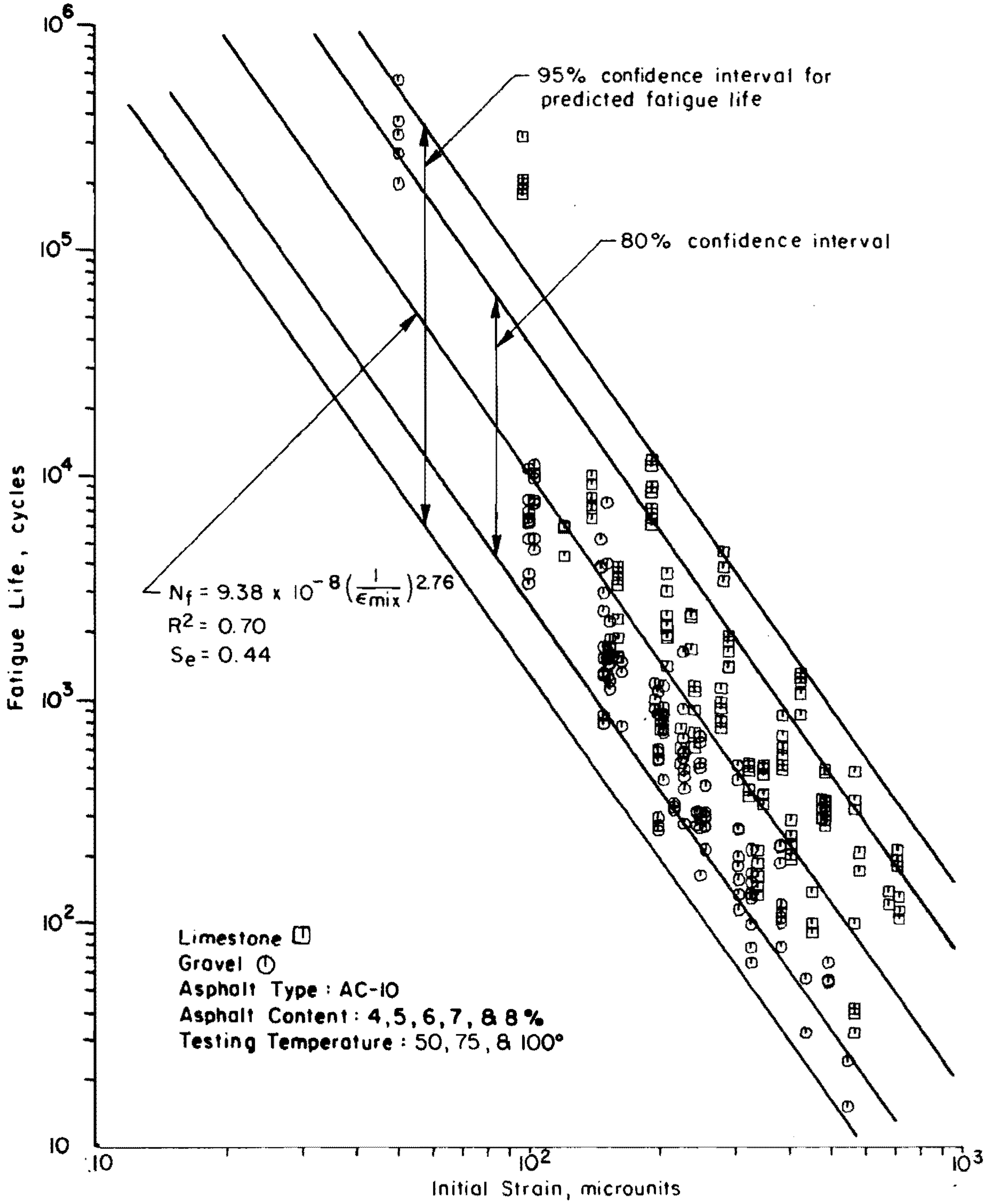


Fig 61. Relationship between fatigue life and initial strain.

75°F:

$$N_f = 8.40 \times 10^{11} \left(\frac{1}{\Delta\sigma}\right)^{3.73}$$

or

$$N_f = 1.56 \times 10^{8} \left(\frac{1}{\sigma_T}\right)^{3.73} \tag{5.15}$$

$(R^2 = 0.72,\ S_e = 0.43)$

100°F:

$$N_f = 1.71 \times 10^{7} \left(\frac{1}{\Delta\sigma}\right)^{2.69}$$

or

$$N_f = 4.14 \times 10^{5} \left(\frac{1}{\sigma_T}\right)^{2.69} \tag{5.16}$$

$(R^2 = 0.92,\ S_e = 0.17)$

It should be noted that all mixtures tested at 50 and 100°F had the same asphalt content, which accounts for the high correlation coefficients.

Furthermore, regression analyses were conducted in an attempt to obtain predictive equations independent of temperature or independent of both temperature and aggregate type, in addition to asphalt content, but these produced very low coefficients of determination. Thus, it appears that there is no universally applicable method of predicting fatigue life on the basis of applied stress or stress difference.

Stress-Strength Ratio. In an attempt to account for mixture and testing differences, the relationship between fatigue life and the stress-strength ratios, which previously has been shown to be a reasonable prediction, was evaluated. The stress-strength ratio was defined as the ratio of the applied repeated stress to the estimated indirect tensile strength presented in Chapter 4.

Results from this study indicate that there was a relatively good correlation between stress-strength ratio and fatigue life. The general relationship is of the form

$$N_f = K_3 \left(\frac{1}{SSR}\right)^{n_3} \tag{5.17}$$

where

N_f = fatigue life,
SSR = stress-strength ratio, in percent, and
K_3 and n_3 = constants.

Separate regression analyses were conducted to determine values for the above constants for each aggregate, asphalt content, and testing temperature. As shown in Table 14, correlation coefficients were reasonably high.

The results obtained are similar in form to those previously obtained for the relationship between fatigue life and initial strain, except for the intercepts of the regression lines. Dividing by tensile strength partially accounts for the effect of temperature and mixture properties. Thus, as with the relationship between fatigue life and initial strain, it is possible to characterize all mixtures by a single relationship between the logarithm of fatigue life and the logarithm of the stress-strength ratio. From regression analyses, the following relationships were obtained:

$$\text{Limestone mixtures:}\quad N_f = 9.14 \times 10^6 \left(\frac{1}{SSR}\right)^{2.86} \tag{5.18}$$

$$(R^2 = 0.74,\ S_e = 0.38)$$

$$\text{Gravel mixtures:}\quad N_f = 5.68 \times 10^7 \left(\frac{1}{SSR}\right)^{3.61} \tag{5.19}$$

$$(R^2 = 0.90,\ S_e = 0.25)$$

In addition, the following overall predictive equation independent of asphalt content, temperature, and aggregate was obtained:

$$N_f = 1.97 \times 10^7 \left(\frac{1}{SSR}\right)^{3.18} \tag{5.20}$$

$$(R^2 = 0.81,\ S_e = 0.35)$$

TABLE 14. VALUES OF CONSTANTS FOR THE LEAST SQUARES REGRESSION EQUATION $N_f = K_3 \left(\frac{1}{SSR}\right)^{n_3}$

(SSR = ratio of repeated stress to indirect tensile strength of mixture)

Temperature, °F	Aggregate*	Asphalt Content, %	Number of Specimens	K_3	n_3	Correlation Coefficient R	Standard Error of Estimate S_e
Individual Mixtures							
50	Limestone	6	9	6.25×10^7	3.74	0.99	0.06
	Gravel	6	9	5.96×10^9	5.19	0.99	0.06
75	Limestone	4	15	1.63×10^6	2.67	0.95	0.14
		5	20	4.66×10^6	2.86	0.98	0.09
		6	27	2.06×10^7	3.19	0.98	0.09
		7	36	3.04×10^8	3.84	0.99	0.14
		8	14	3.02×10^8	3.33	0.99	0.07
	Gravel	4	14	5.69×10^6	3.06	0.97	0.11
		5	21	4.82×10^6	2.96	0.97	0.10
		6	28	3.21×10^7	3.46	0.97	0.13
		7	40	2.65×10^8	4.11	0.96	0.27
		8	14	2.05×10^7	3.13	0.91	0.22
100	Limestone	6	9	3.79×10^6	2.66	1.00	0.06
	Gravel	6	9	4.28×10^6	2.72	0.99	0.10
Combined Mixtures							
50	Ls & gr	6	18	1.99×10^8	4.10	0.97	0.12
75	Limestone	4,5,6,7,& 8	112	1.16×10^7	2.92	0.85	0.40
	Gravel	4,5,6,7,& 8	117	8.66×10^7	3.76	0.96	0.24
	Ls & gr	4,5,6,7,& 8	229	2.43×10^7	3.25	0.89	0.37
100	Ls & gr	6	18	4.32×10^6	2.71	0.99	0.08
50,75,& 100	Limestone	4,5,6,7,& 8	130	9.14×10^6	2.86	0.86	0.38
	Gravel	4,5,6,7,& 8	135	5.68×10^7	3.61	0.95	0.25
	Ls & gr	4,5,6,7,& 8	265	1.97×10^7	3.18	0.90	0.35

*Ls - limestone, gr - gravel.

Figure 62 shows the position of Eq 5.20 relative to all data points. From Figure 62 it can be seen that most of the points lie entirely within the 90 percent confidence interval. Furthermore, all the points are above the lower bound of the 90 percent confidence interval, which indicates that the line could possibly be used to predict minimum fatigue life. Thus, using the same n_3 value, K_3 was estimated to obtain an approximate minimum fatigue life predictive equation:

$$N_f \text{ (min)} = 5.23 \times 10^6 \left(\frac{1}{SSR} \right)^{3.18} \tag{5.21}$$

For comparison, the average fatigue life predictive equations obtained from earlier studies (Refs 52 and 55) were superimposed on data from this study (Fig 63), from which it can be seen that both lines fell fairly well within the data points.

Results from this study indicated that prediction of fatigue life in terms of stress-strength ratio is as good as that in terms of initial strain. Like initial strain $\frac{\sigma_T}{E_S}$, stress-strength ratio is easy to estimate and does not involve fatigue loading. Since estimation of tensile strength only requires the measurement of failure load and does not require deformation measurements, this relationship probably has more application with respect to use by the district laboratories.

Relationship with Modulus of Elasticity

Previous studies have shown that fatigue life is related to mixture stiffness and that factors which affect stiffness also have a similar effect on controlled-stress fatigue life. In this study stiffness was evaluated in terms of the static modulus of elasticity and the initial resilient modulus of elasticity, estimated from the relationship between resilient modulus and the number of load applications by extrapolating to the first load application.

As in previous studies, it was generally found that fatigue life tended to increase with increased stiffness and that factors which affected the modulus, e.g., asphalt content and testing temperature, had a similar effect on fatigue life. However, fatigue life cannot be estimated from moduli.

Mixture Properties

Two mixture properties which are easily measured and which have been

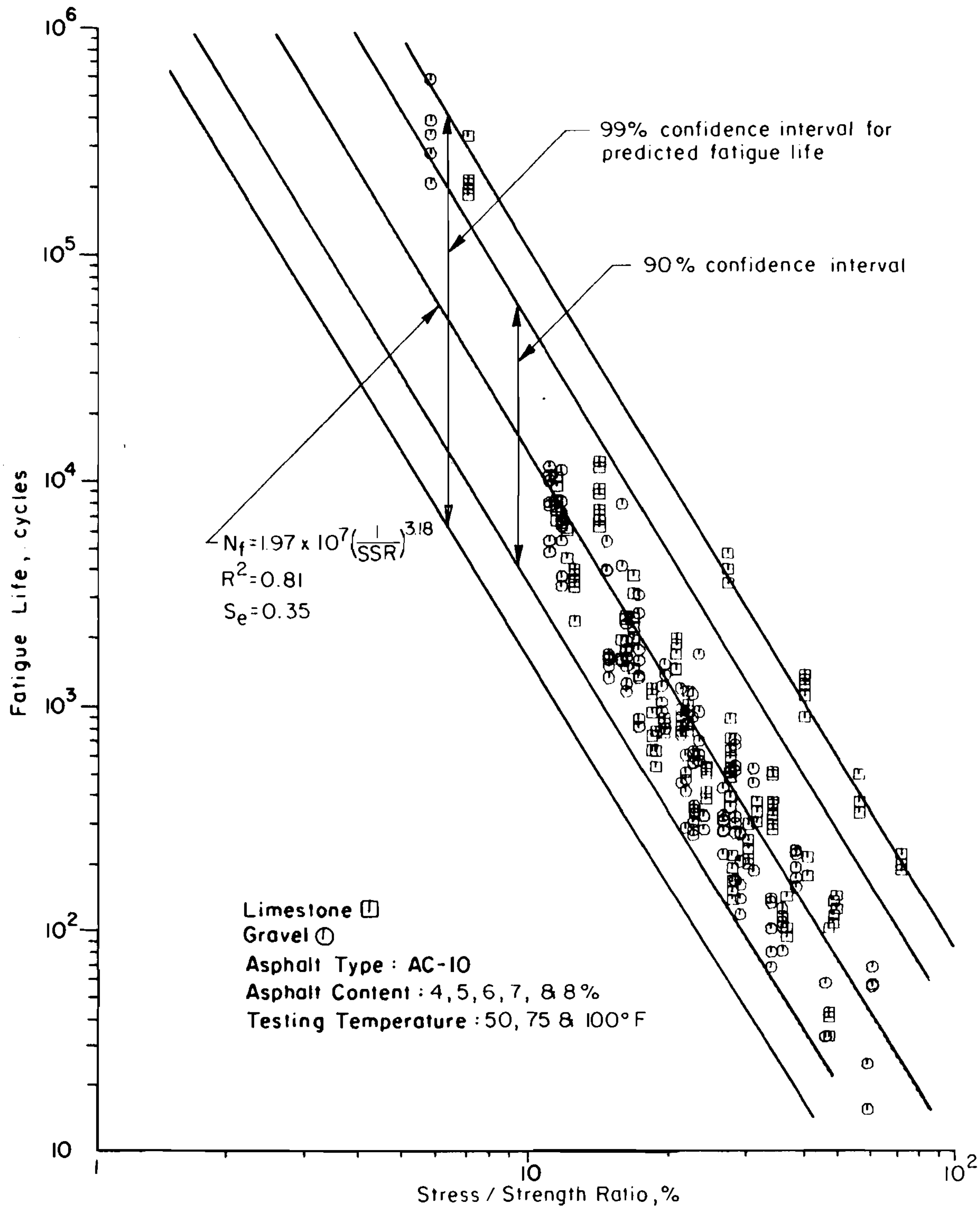


Fig 62. Relationship between fatigue life and stress-strength ratio.

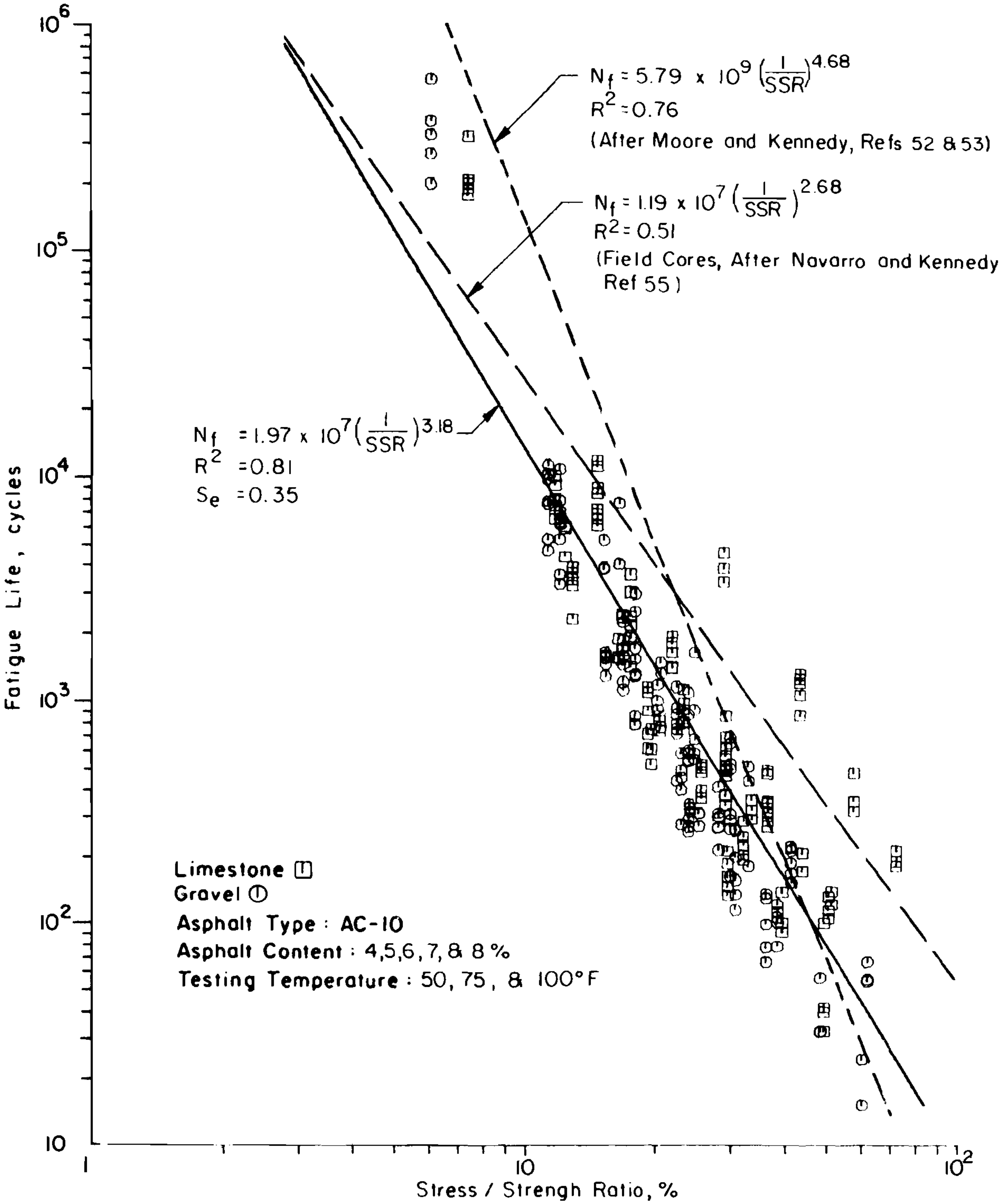


Fig 63. Comparison of various relationships between fatigue life and stress-strength ratio.

related to fatigue life are air void content and density.

Relationship with Air Void Content. The effect of air void content on the fatigue life of both limestone and gravel mixtures tested at 75°F was investigated for each stress level (Appendix E).

Many investigators, including Saal and Pell (Ref 67), Pell and Taylor (Ref 62), Epps and Monismith (Ref 9), and Bazin and Saunier (Ref 4) in stress-controlled tests; and Santucci and Schmidt (Ref 68) in controlled-strain tests, have indicated that fatigue life of mixtures increases with decreasing air void content. However, utilizing the indirect tensile test method in a controlled-stress test, Moore and Kennedy (Ref 52) reported that air void content was not directly related to laboratory tensile fatigue life.

In this study, it appeared that an optimum air void content for maximum fatigue life existed (Appendix E). For air void contents less than about 3 percent, there was an increase in fatigue life with an increase in air voids. However, for air void contents greater than about 3 percent, a further increase in air voids resulted in a decrease in fatigue life.

In general, a decrease in air void content can be obtained in various ways, including (1) an increase in compactive effort, (2) an increase in asphalt content, (3) change from open to dense gradation, and (4) combinations of (1), (2), and (3) above. In this study, the compactive effort and the gradation were held constant so that changes in air void content were dependent on the changes in asphalt content (Fig 27), and thus the apparent effect of air void content was judged to be due to the effect of asphalt content. This is illustrated by the fact that the approximate optimum asphalt content of about 6 percent for maximum fatigue life corresponds to an air void content of about 3 percent (Fig 27), also observed for maximum fatigue life.

Additional information would be required in order to arrive at any definite conclusion concerning the effects of air void content but it is felt that any apparent relationship between fatigue life and air void content is probably due to the fact that both air void content and fatigue life are related to other factors.

Relationship with Density. Density and air void content are mixture properties that are inversely related to each other. Mostly, researchers have preferred to correlate fatigue life with air void content instead of density. Considering (1) that not much uniformity has been achieved in correlating fatigue life with air void content; (2) that, as Epps and Monismith suggested, besides absolute volume, the structure (size, shape, and degree of interconnection) of air void is important (Ref 9), a condition that is not easy to evaluate; and (3) that it is a lot easier to estimate density than air void, it seems that efforts should be made to correlate fatigue life with density rather than air void.

In this study, an approximate linear relationship was found to exist between the logarithm of fatigue life and bulk density for tests conducted at 75°F, as follows:

$$\log N_f = m\gamma + C \qquad (5.22)$$

where

N_f = fatigue life, cycles,

γ = bulk density, pcf,

m = slope of log fatigue life versus bulk density, a constant depending on stress level, and

C = intercept of line, constant, depending on stress level.

The values of m and C obtained from regression analysis are shown in Table 15, from which it can be seen that for individual and combined aggregates, m ranged between 0.10 and 0.17 while C ranged between -19.0 and -11.9. The relationships between the logarithm of fatigue life and bulk density for combined aggregates are illustrated for each stress level in Fig 64. Due to the relatively high correlation coefficients of the relationships for each stress level, an attempt was made to find a relationship independent of stress level within the stress range 16 psi to 40 psi. This yielded the following relationship:

$$\log N_f = 0.137\gamma - 16.73, \ (R = 0.57,\ S_e = 0.51) \qquad (5.23)$$

Equation 5.23 was obtained with a rather low correlation coefficient of 0.57 and a standard error of estimate of 0.51. Thus, it would seem that a fairly

TABLE 15. VALUES OF CONSTANTS FOR THE LEAST SQUARES REGRESSION EQUATION $\log N_f = m\gamma + C$

Temperature, 75° F

Aggregate	Stress Level, psi	Number of Specimens	C	m	Correlation Coefficient R	Standard Error of Estimate S_e
Limestone	16	25	-17.22	0.145	0.85	0.20
	24	27	-18.87	0.153	0.90	0.18
	32	24	-11.89	0.101	0.81	0.18
	40	27	-13.89	0.113	0.81	0.20
Gravel	16	27	-20.32	0.168	0.86	0.24
	24	31	-18.35	0.149	0.89	0.21
	32	28	-18.27	0.146	0.90	0.19
	40	26	-18.28	0.145	0.89	0.20
Combined limestone & gravel	16	52	-17.30	0.146	0.84	0.23
	24	58	-18.24	0.148	0.90	0.20
	32	52	-15.36	0.126	0.87	0.19
	40	53	-15.16	0.122	0.85	0.21
Limestone	All	103	-15.77	0.130	0.51	0.51
Gravel	All	112	-19.05	0.154	0.62	0.51
Combined limestone & gravel	All	215	-16.73	0.137	0.57	0.51

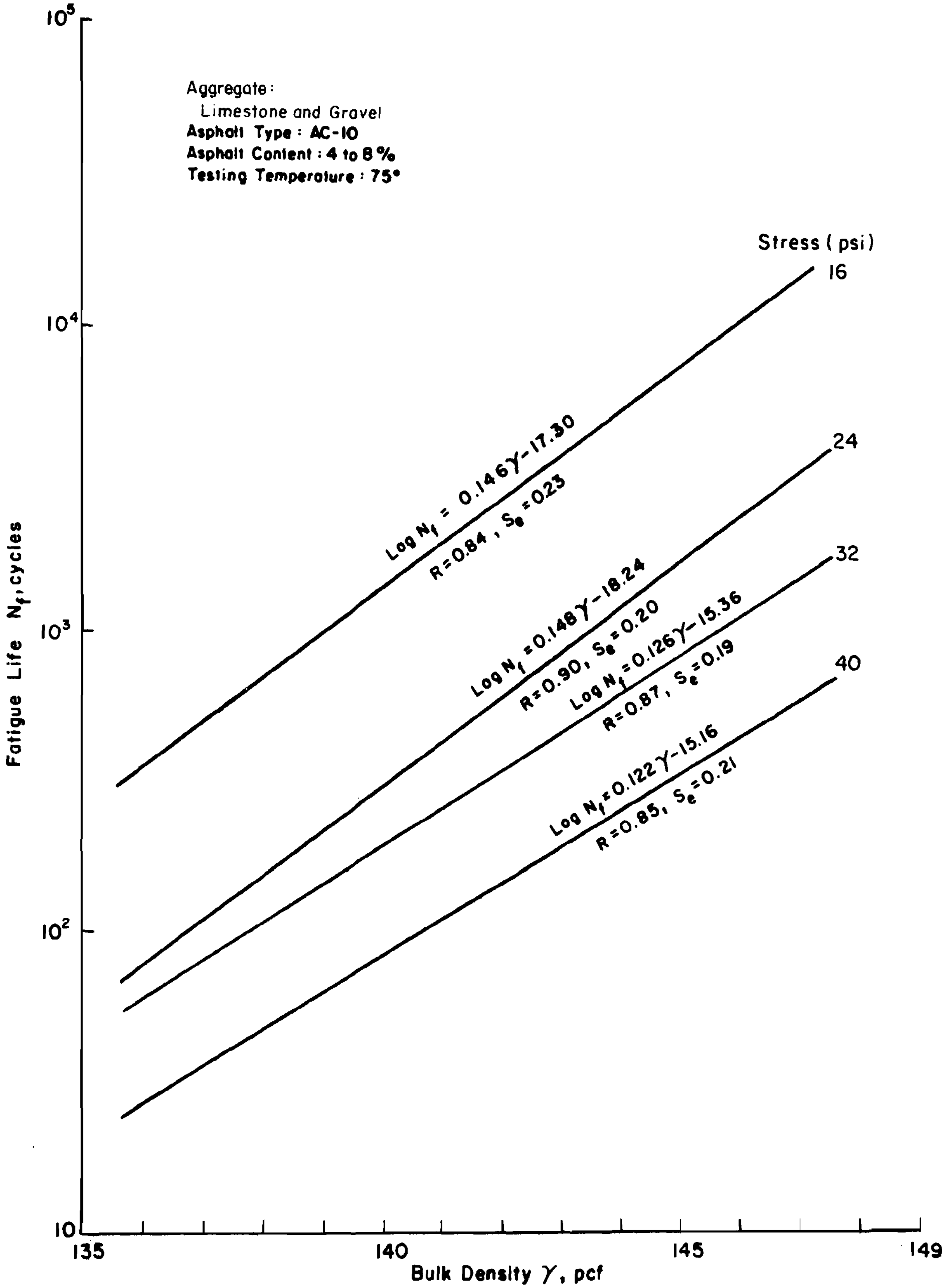


Fig 64. Relationships between bulk density and fatigue life.

good relationship exists between bulk density and the logarithm of fatigue life and that the relationship is stress dependent.

Additional work is needed before any conclusions are made regarding the relationships between fatigue life and density, especially since the range of variables, including the range of density values, utilized in this study is relatively small.

Selection of Basis of Prediction Equation

Any equation attempting to predict fatigue life must be a function of several variables. Three of these variables are (1) stress level σ , (2) mixture properties M , and (3) test temperature T . Thus, the fatigue equation can be written in the form

$$N_f = f(\sigma, M, T) \tag{5.24}$$

where

M = f(aggregate type and gradation; asphalt type and amount; mixing, compaction, and test temperature; compactive effort, curing conditions, etc.)

Results from this study indicate that N_f can be predicted fairly accurately in three ways:

(1) Stress vs N_f relationship: In this case, the constants K_2 and n_2 are functions of mixture properties and they must be correlated with important mixture properties; otherwise, they must be estimated for each mixture.

(2) Strain vs N_f (where strain = $\frac{\text{dynamic stress}}{\text{static modulus of elasticity}}$): This gives a relationship between strain and fatigue life which is fairly independent of temperature, the properties having been, to some extent, eliminated by dividing stress by static modulus.

(3) Stress-strength ratio vs N_f: In this case, the mixture properties and test temperature are to some extent accounted for by the use of the tensile strength. Thus, the use of this relationship should be fairly independent of mixture properties and the constants of the equation should be approximately uniform. However, more work is still needed in this direction.

Each of the above three methods is simple and does not require long term fatigue tests. This is a desirable property of a fatigue life prediction equation. A final choice will depend on available information and the accuracy required.

CHAPTER 6. EFFECTS OF REPEATED LOADS ON LOAD-DEFORMATION CHARACTERISTICS

Previous studies have indicated that strain increases and modulus decreases as a result of repetitions of load. In this study, the effects of repeated indirect tensile stress on strain, modulus of elasticity, and Poisson's ratio were examined and the results are summarized in this chapter.

STRAIN

The effects of repeated loads on the following four types of strain were evaluated:

(1) total resilient strain, based on total recovered deformation per cycle;

(2) instantaneous resilient strain, based on instantaneous recovered deformation per cycle;

(3) individual total strain, based on total deformation per cycle; and

(4) permanent strain, based on cumulative total permanent deformation.

Total Resilient Strain

The effect of repeated stress applications on total resilient strain is illustrated by the typical relationship for a limestone mixture containing 6 percent asphalt tested at a stress level of 24 psi and at 75°F as shown in Fig 65. The number of load applications is expressed as a percent of fatigue or fracture life. An approximately linear relationship existed between total resilient strain and number of load applications, up to about 60 or 70 percent of the fracture life, at which time resilient strain increased more rapidly until failure, or fracture, occurred. This point of excessive increase in resilient strain corresponded to the failure zone of the specimen, at which time substantial cracking was evident from visual inspection.

The shape of the curve in Fig 65 is similar to that presented by Papazian and Baker for the center deflection of an asphalt concrete beam subjected to repeated flexure (Ref 56).

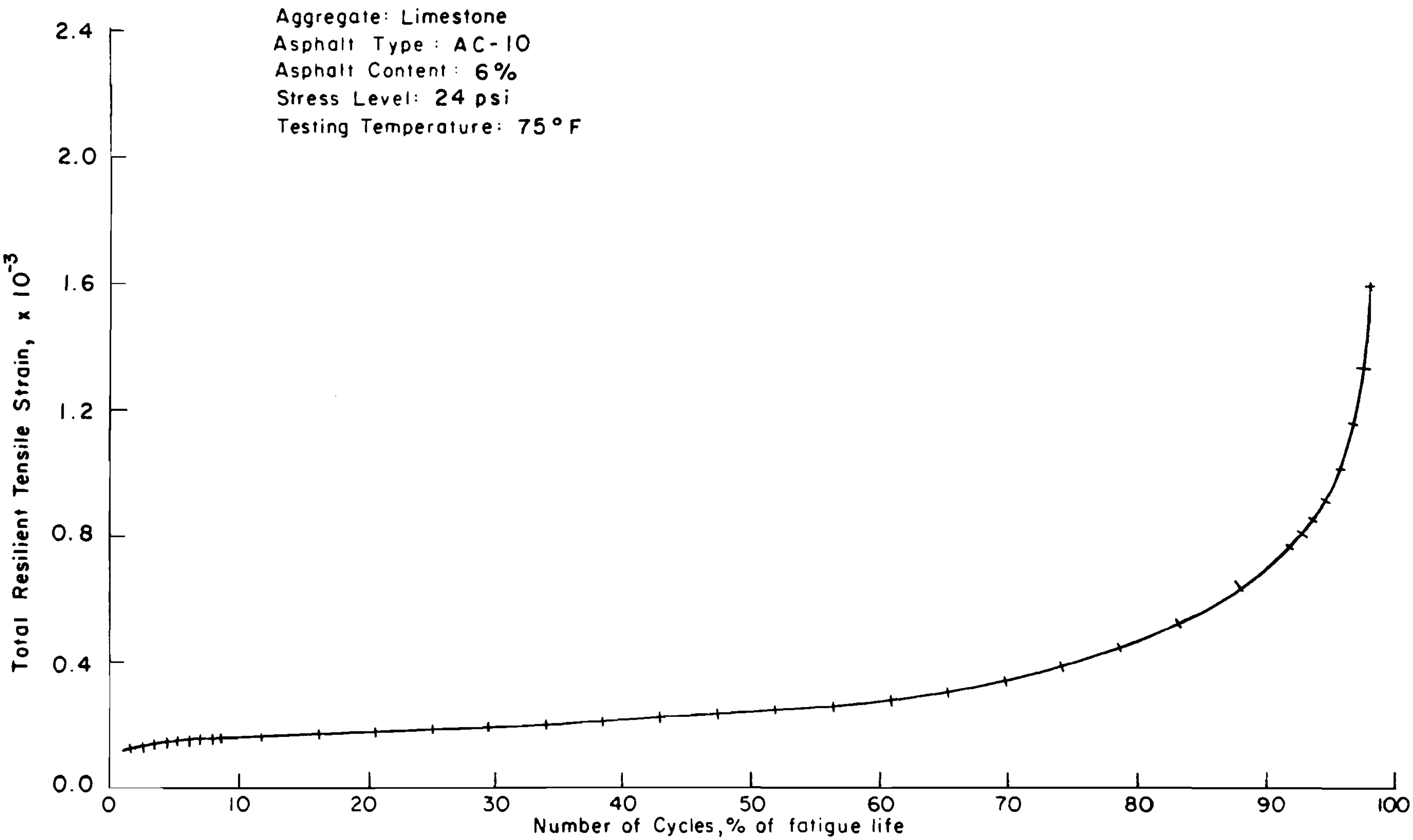


Fig 65. Effect of repeated loads on total resilient tensile strain.

Instantaneous Resilient and Individual Total Strains

The relationships between both instantaneous and individual total tensile strains and the number of load applications were similar to the total resilient strain relationships. A comparison of the three strain types evaluated from a single specimen is illustrated in Fig 66. The only difference between the relationships is the relative position of the curves. The values of the strains are proportional to their respective deformations, the basis on which the strains were estimated, with the individual total strains having the largest values and the instantaneous resilient strains the smallest.

Results from this study indicated that the instantaneous or total resilient strains and the individual total strains of asphalt mixtures increased slightly with increasing load applications during the useful life of the specimen (about 70 percent of fracture life) and then increased very rapidly with further applications of load.

Permanent Strain

Results from this study indicated that the behavior of the horizontal and vertical permanent strains of asphalt mixtures due to repeated loads can be divided into three distinct zones. As illustrated in Figs 67 and 68, which show typical relationships between horizontal or vertical permanent strains and number of cycles for both limestone and gravel mixtures, the three zones are as follows:

(1) Zone of initial adjustment to load: This consists of the first 10 percent of the fracture life and a slight curvature is exhibited. In this zone, the specimen is probably adjusting to load and also may be undergoing some additional compaction.

(2) Zone of stable condition: This consists of between 10 percent and 70 percent of the fracture life, during which the permanent strain exhibits a linear relationship with the number of load repetitions. This can also be termed as the useful life of the specimen with respect to pavement rutting (manifestation of permanent deformation).

(3) Failure zone: This extends from about 70 percent of fracture life to the instant of complete fracture (100 percent). This zone, as presented earlier, also corresponds to the region of excessive resilient strain and it can therefore be suggested that the specimen is experiencing all forms of load associated distress.

Similar relationships between permanent deformation and load applications have been obtained by a number of investigators, including Deacon (Ref 6) and McElvaney (Ref 40), as discussed in Chapter 2. In addition, in a recent study

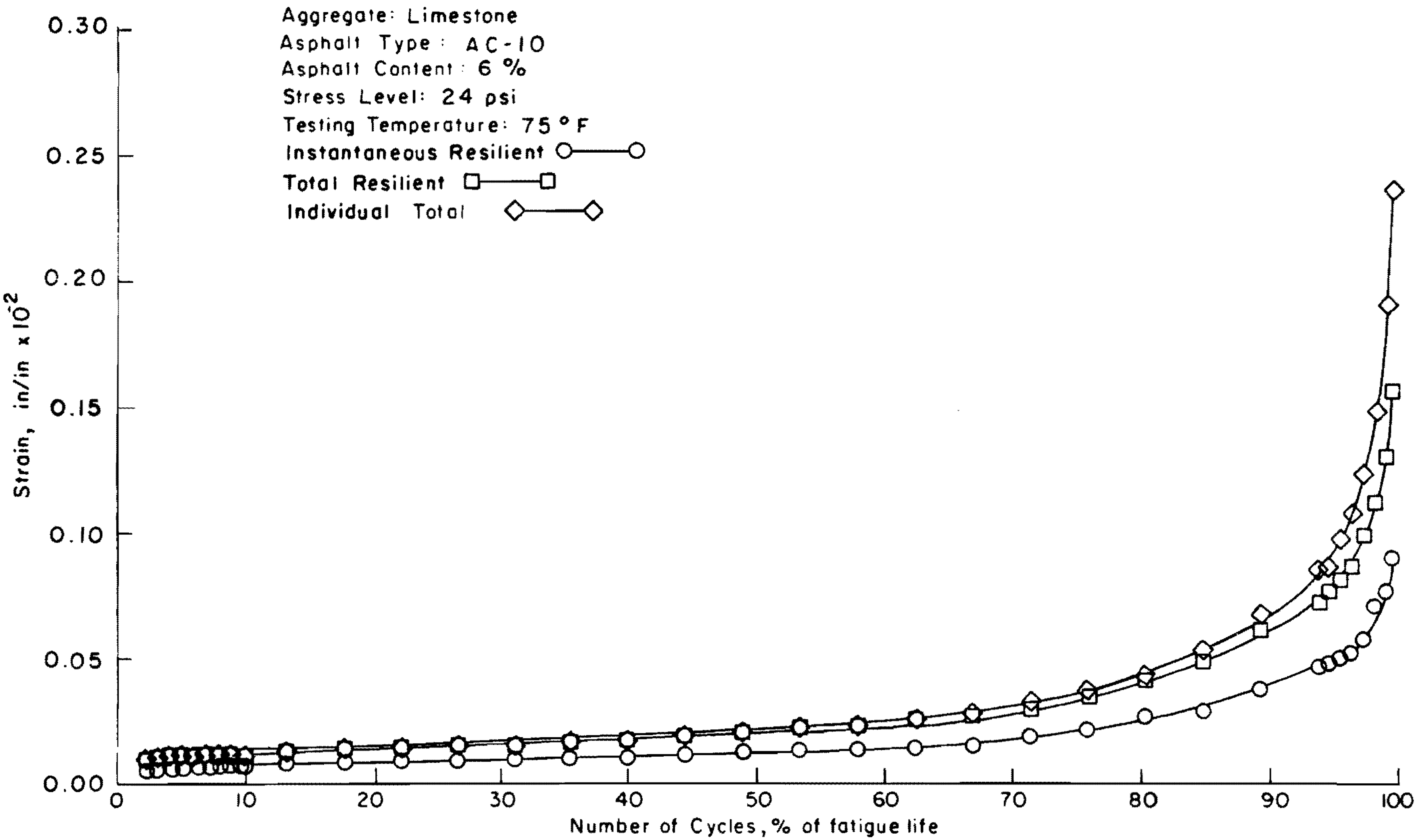


Fig 66. Comparison of instantaneous resilient, total resilient, and individual total tensile strains.

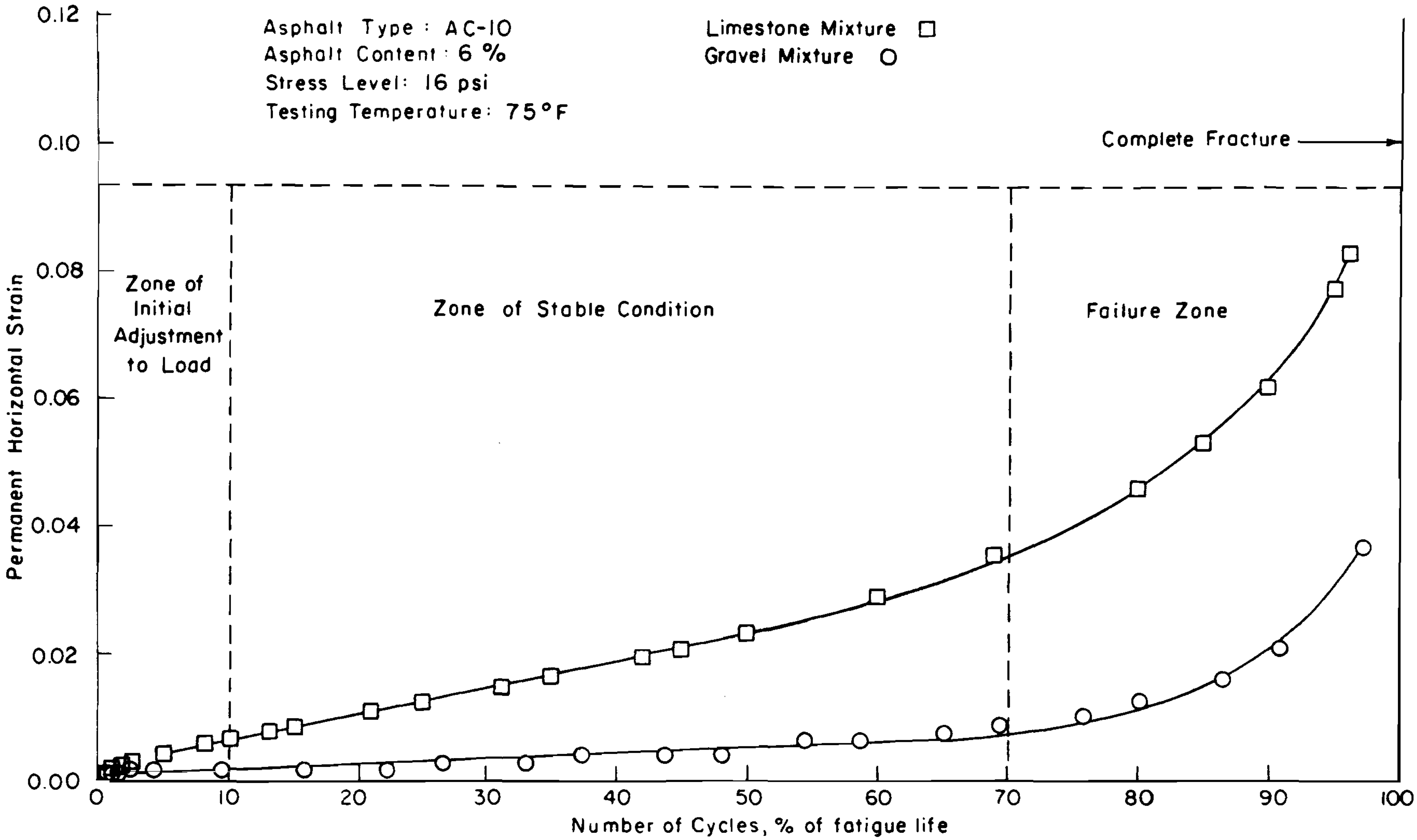


Fig 67. Effects of repeated loads on horizontal permanent strain.

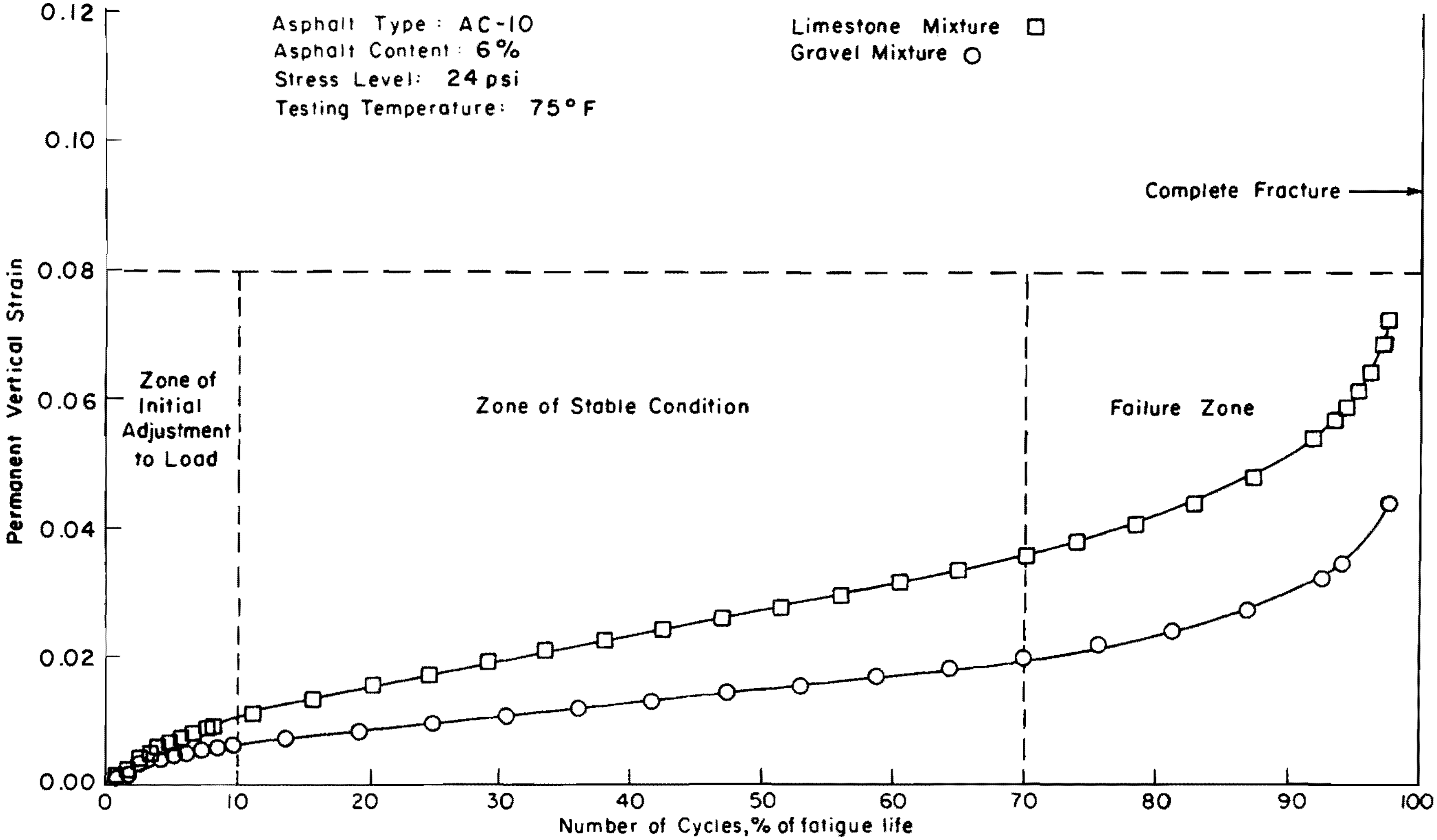


Fig 68. Effects of repeated loads on vertical permanent strain.

(Ref 5), Cowher and Kennedy recognized the above three zones for the relationship between horizontal permanent deformation and number of load applications.

MODULUS

The effects of repeated loads on various forms of modulus estimated as described earlier in Chapter 3 were investigated. These include:

(1) instantaneous resilient modulus, based on instantaneous resilient strains;

(2) total resilient modulus, based on total resilient strains;

(3) modulus of individual total deformation, based on individual total strains; and

(4) modulus of cumulative total deformation, based on cumulative total strains.

Variations of Instantaneous Resilient Moduli, Total Resilient Moduli, and Moduli of Individual Total Deformation with Load Repetitions

The effects of repeated-loads on the total resilient modulus are illustrated for a typical specimen in Fig 69. The figure indicates the variation of resilient modulus with load applications for a gravel-mix specimen containing 7 percent asphalt and tested at 32 psi. For comparison, the corresponding values of instantaneous resilient modulus and modulus of individual total deformation for the same specimen are plotted as shown in Fig 70 for the gravel mixture and in Fig 71 for a limestone mixture. The relationships, as for permanent strains, can be divided into three zones: (1) Conditioning zone - the initial portion of the curves, up to about 10 percent of the fracture life. The shape of the curve here is not definite, ranging from concave up to concave down. This zone corresponds to the zone of initial adjustment of specimen to load, as explained earlier for permanent strain. (2) Stable zone - the portion of the curves between 10 percent and about 80 percent of the fracture life. This portion, which is approximately linear, with a gradual decrease in modulus with increasing applications of load, also represents the useful life of the specimen. (3) Failure zone - the last 20 percent of the fracture life, in which the curve undergoes a sharp curvature, with a rapid drop in modulus.

While the shapes of the curves are similar (Figs 70 and 71), the positions differ, with the instantaneous resilient moduli having the largest

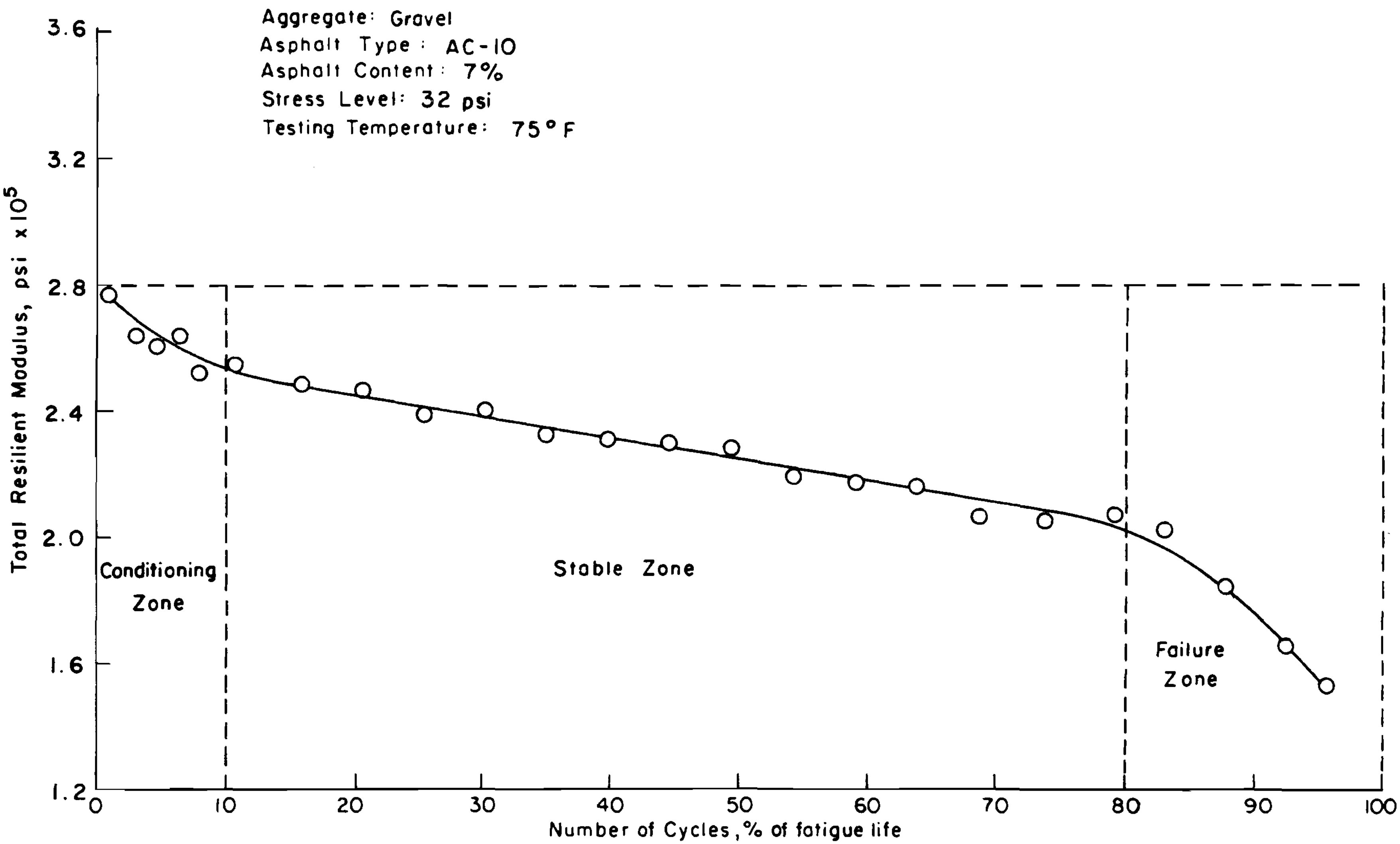


Fig 69. Effect of repeated loads on total resilient modulus.

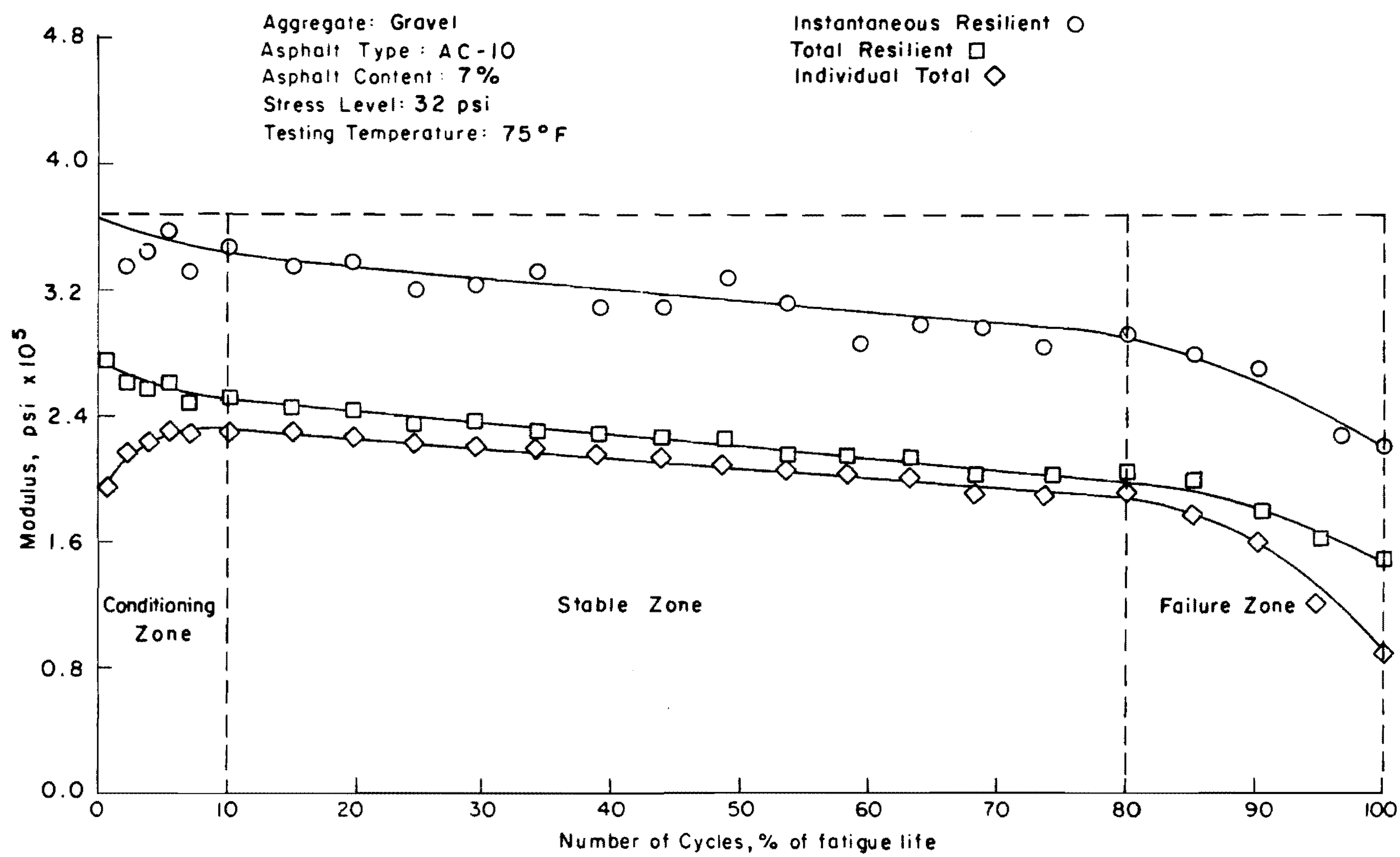


Fig 70. Comparison of instantaneous resilient, total resilient, and individual total modulus for gravel mixtures.

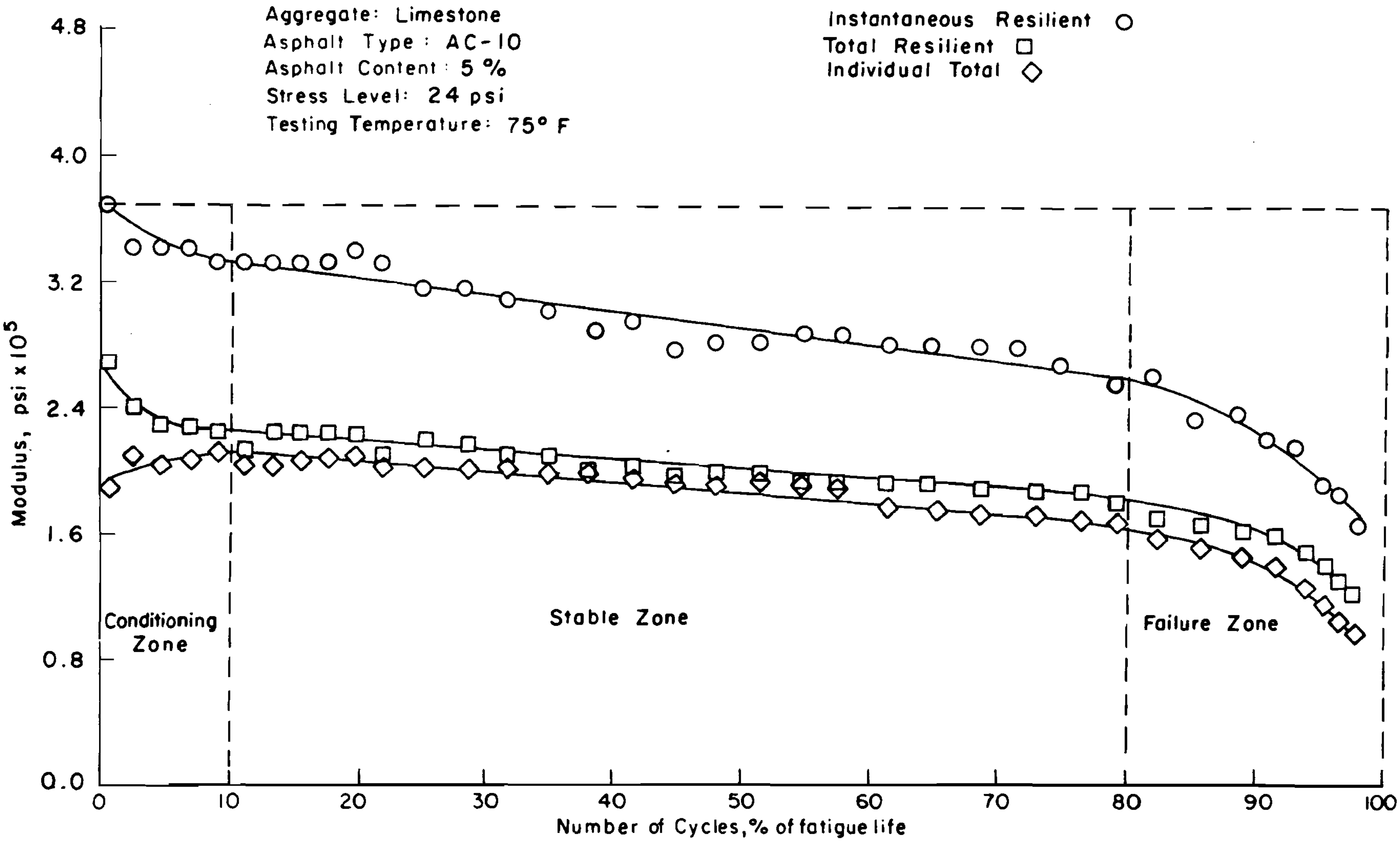


Fig 71. Comparison of instantaneous resilient, total resilient, and individual total modulus for limestone mixtures.

values and the moduli of individual total deformation having the lowest values. The above pattern is typical of all specimens tested.

Deterioration of Instantaneous and Total Resilient Moduli

Linear regression analyses were conducted to establish the relationship between resilient moduli and number of cycles for the linear portions of the relationships, between 10 and 80 percent of fracture life. The results obtained for individual specimens are contained in Appendix F, which indicates that correlation coefficients are relatively high, supporting the assumption of linearity of the stable zone.

Average slope values for each testing condition together with coefficients of variation are presented in Table 16 for both instantaneous and total resilient moduli. The mean slopes ranged between 7 and 3,005 psi/cycle for the instantaneous resilient moduli and between 5 and 991 psi/cycle for the total resilient moduli. These values are exploratory in nature since no such values had ever been reported. Also, due to the limitations of factors considered and the high variations in the slopes, the values can at best serve only as a guideline and provide general estimates of the magnitude of the deterioration of the resilient modulus with load repetitions.

There seemed to be a minimum slope value at an optimum asphalt content of about 6 percent, as illustrated in Figs 72 and 73. This is approximately the same asphalt content as the optimum for maximum fatigue life, which indicates that longer fatigue lives are associated with smaller rates of decay of instantaneous and total resilient moduli.

The effect of stress on slopes is illustrated in Figs 74 and 75, which indicate that the slopes, or deterioration, increased with increased stress.

The effect of testing temperature is also indicated in Figs 74 and 75 for stress levels of 16 and 24 psi and for mixtures containing 6 percent asphalt and tested at 75 and 100°F. The figures indicate larger slopes for higher temperatures.

The gravel mixtures generally had larger slopes than the limestone mixtures except for mixtures containing 7 percent asphalt at stresses larger than about 24 psi, in which case the limestone mixtures had larger slopes than the gravel mixtures (Figs 72 through 75). This was true for both instantaneous and total resilient moduli.

TABLE 16. AVERAGE VALUES OF THE SLOPE OF THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND LOAD REPETITIONS

Test Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Specimens	Instantaneous Resilient Modulus		Total Resilient Modulus	
					Mean Slope, psi/cycle	CV, %	Mean Slope, psi/cycle	CV, %
50	Limestone	6	72	3	20	71	14	69
			96	2	7	10	21	3
			120	3	252	94	162	88
	Gravel	6	72	2	79	52	52	54
			96	3	373	72	291	65
			120	3	530	84	487	36
75	Limestone	4	24	3	464	52	298	34
		5	16	3	22	116	18	56
			24	3	131	56	76	30
			32	3	251	69	162	53
			40	3	540	46	359	24
		6	16	5	7	133	5	80
			24	3	17	29	14	14
			32	4	28	31	31	16
			40	5	175	57	129	29
		7	16	5	27	113	9	89
			24	3	118	27	81	26
			32	4	359	52	247	33
			40	3	432	48	280	19
		8	16	2	31	81	16	38
			24	5	91	31	51	54
	Gravel	4	24	3	631	34	602	80
		5	16	3	111	5	56	15
			24	3	223	91	181	51
			32	3	858	12	496	21
			40	3	1476	21	867	22
		6	16	4	13	177	9	144
			24	4	39	67	41	51
			32	5	77	22	64	33
			40	5	222	41	182	30
		7	16	6	33	79	20	75
			24	7	183	108	109	86
			32	5	285	126	197	114
			40	3	347	28	297	25
		8	16	1	196	–	85	–
			24	3	308	49	200	44
100	Limestone	6	8	3	60	118	9	89
			16	3	135	21	81	31
			24	3	584	77	379	29
	Gravel	6	8	3	81	42	58	28
			16	3	627	22	317	4
			24	3	3005	55	991	36

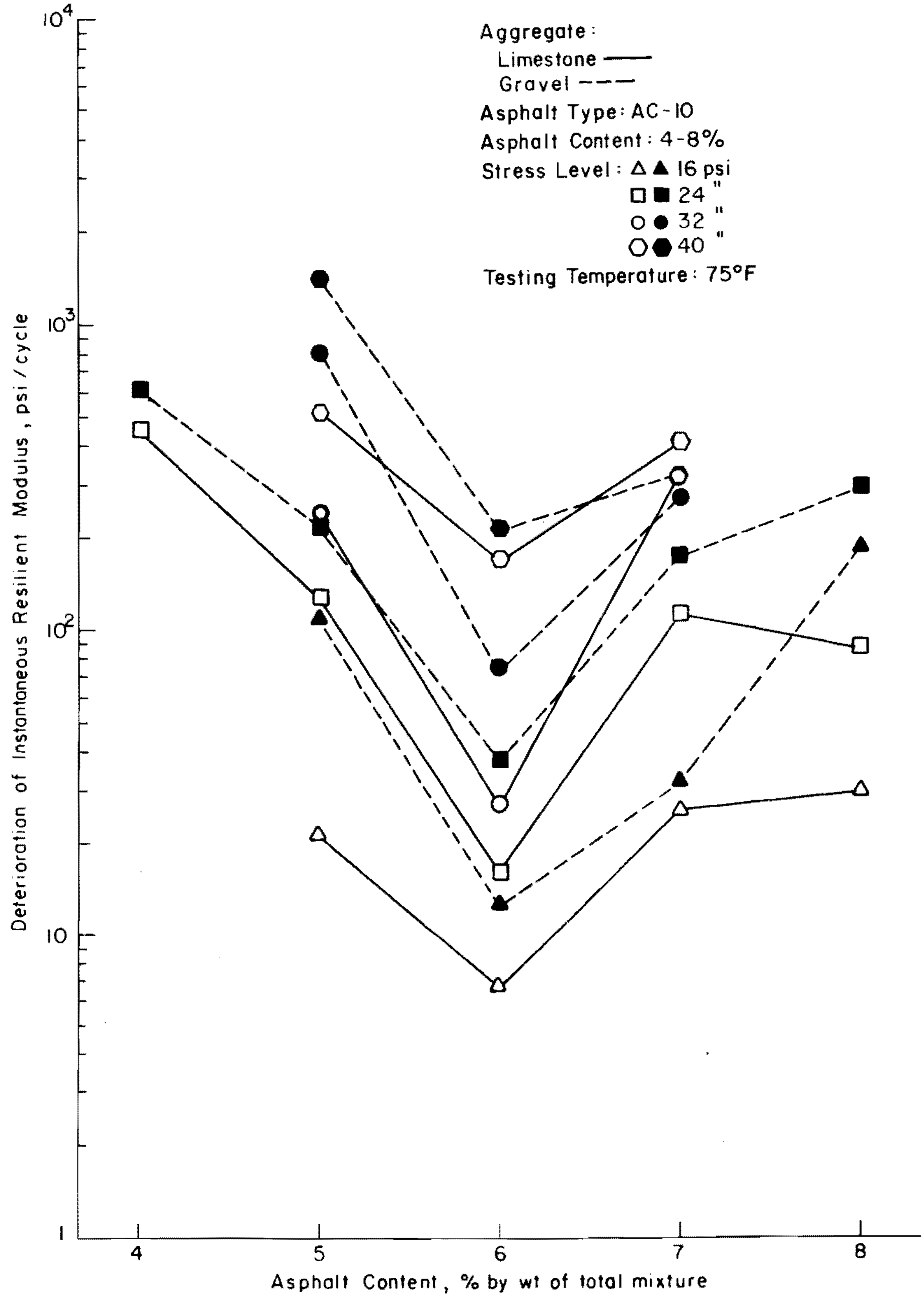


Fig 72. Effect of asphalt content on deterioration of instantaneous resilient modulus.

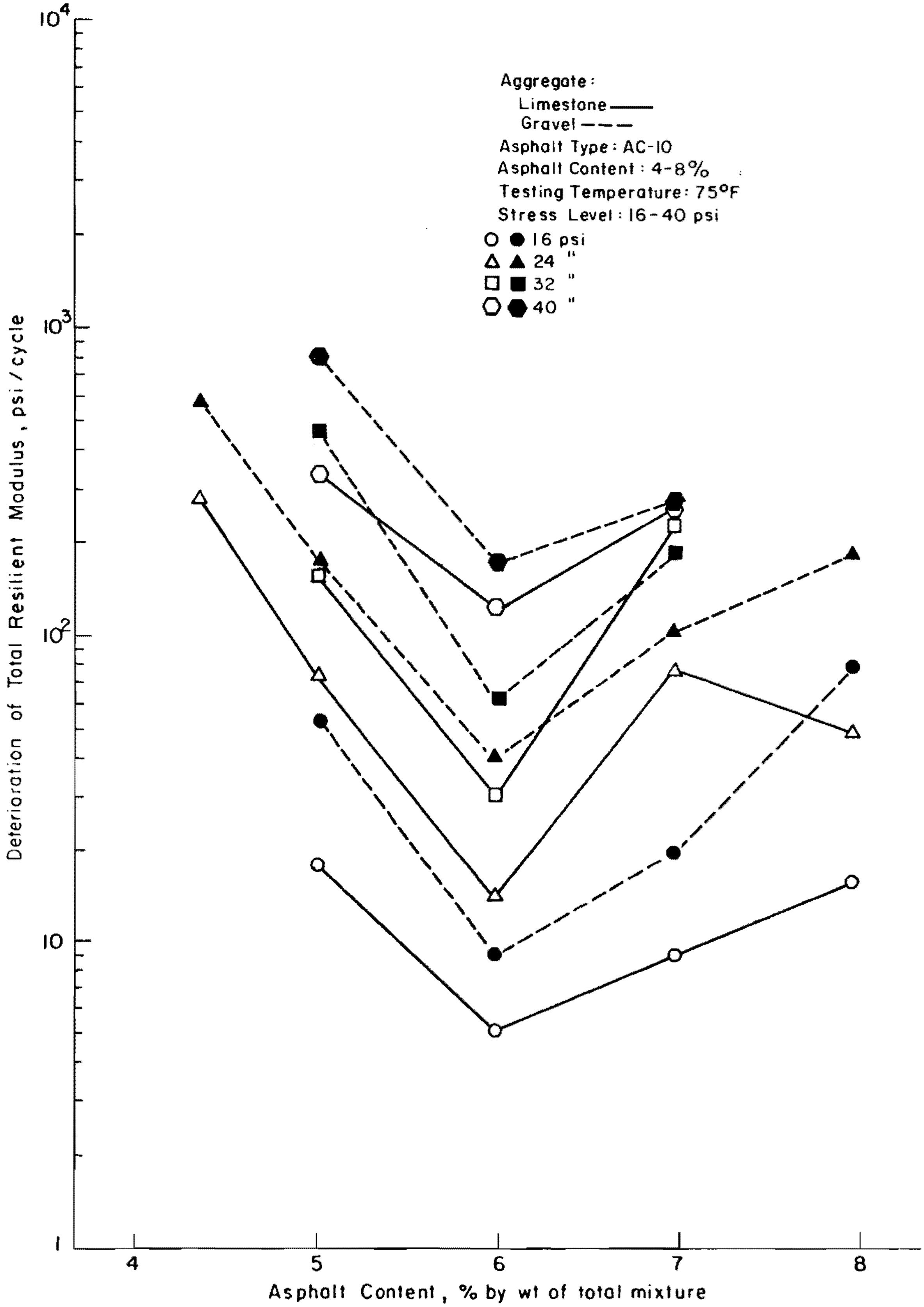


Fig 73. Effect of asphalt content on deterioration of total resilient modulus.

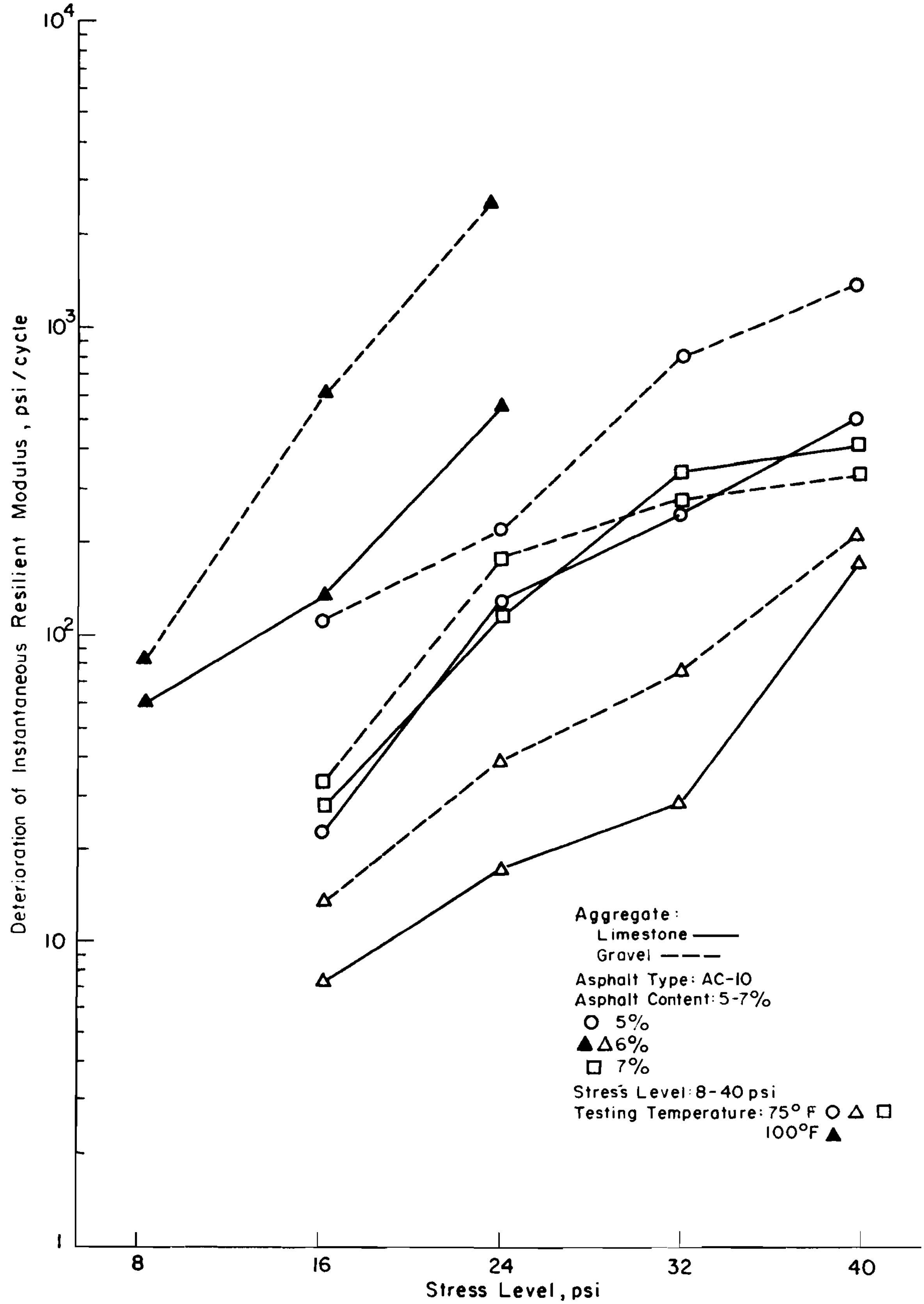


Fig 74. Effect of stress on deterioration of instantaneous resilient modulus.

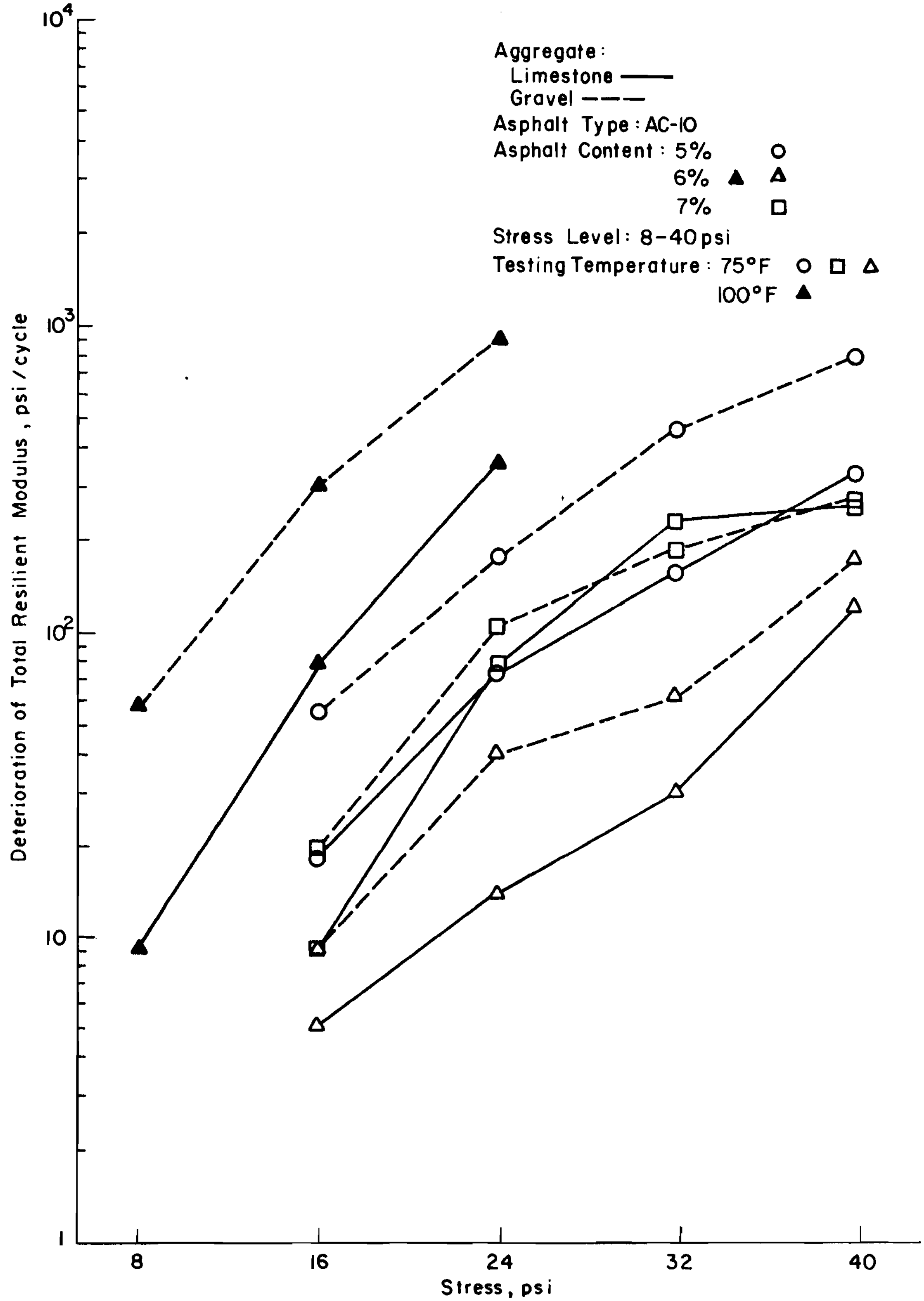


Fig 75. Effect of stress on deterioration of total resilient modulus.

From the above observations, it seems that factors which affect fatigue life also affect the deterioration of resilient modulus.

Average Values of Repeated-Load Resilient Moduli

In this study, it was partly intended to estimate values of dynamic modulus that can be used for design purposes. Accordingly efforts were made to arrive at such values by giving consideration to the portion of the modulus versus number of cycle relationships lying between 10 and 80 percent of fracture life. Since the extremes at 10 and 80 percent would overestimate or underestimate the modulus, an average value for approximately the middle of the relationships, which represents the stable or useful life of the specimens, was considered. The values were estimated by averaging values at 30, 40, 50, 60, and 70 percent of fracture life. Such values obtained for the individual and total resilient modulus for individual specimens are presented in Appendix G. The means and coefficients of variation for each stress level, asphalt content, aggregate type, and testing temperature are summarized in Table 17.

The mean values of the instantaneous resilient modulus ranged between 126,000 psi and 918,000 psi while those for the total resilient modulus ranged between 91,000 psi and 802,000 psi. The coefficients of variation for the total resilient modulus were generally slightly less than those for the instantaneous resilient modulus.

The repeated-load resilient modulus values presented in Table 17 compared reasonably well with values obtained for various mixtures in studies completed elsewhere. Such values obtained from Refs 31 and 48 are presented in Table 18 for comparison.

Both instantaneous and total resilient modulus are fair measures of elasticity of the mixtures and can therefore be used for design purposes. However, the measurement of instantaneous resilient deformation is subject to more error than that of total resilient deformation because instantaneous resilient deformations are relatively smaller than total resilient deformations, and since these deformations are the respective basis on which the modulus values were estimated, the total resilient modulus may be subject to less error than the instantaneous resilient modulus. This in part accounts for the slightly smaller values of coefficients of variation.

TABLE 17. SUMMARY OF REPEATED-LOAD MODULUS

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Specimens	Modulus of Elasticity: Instantaneous Resilient Mean, psi x 10^3	Instantaneous Resilient CV, %	Total Resilient Mean, psi x 10^3	Total Resilient CV, %	Cumulative Total Mean, psi x 10^3	Cumulative Total CV, %
50	Limestone	6	72	3	736.3	4.5	609.4	8.3	23.5	14.0
			96	3	641.6	2.8	532.3	4.5	41.9	18.4
			120	3	775.1	3.0	609.7	4.1	45.5	8.1
			Total	9	717.7	8.8	583.8	8.4	37.0	30.3
	Gravel	6	72	3	918.4	17.0	802.5	11.8	42.2	12.3
			96	3	893.5	17.1	758.1	19.4	57.6	8.2
			120	3	828.0	10.9	663.1	4.2	76.6	6.7
			Total	9	880.0	14.2	741.2	14.6	58.8	26.4
75	Limestone	4	24	3	260.1	11.5	183.2	8.8	14.6	7.5
		5	16	3	350.8	4.4	232.6	3.1	7.8	10.3
			24	3	299.6	10.7	197.5	5.4	8.9	3.4
			32	3	362.8	19.8	244.2	14.1	12.8	8.6
			40	3	356.0	3.6	251.2	3.0	15.1	7.3
			Total	12	342.3	12.7	231.4	11.6	11.1	28.8
		6	16	5	276.6	43.5	183.2	29.3	3.2	12.5
			24	5	346.1	29.1	233.1	25.8	4.7	6.4
			32	5	267.0	2.8	186.5	2.5	6.7	6.0
			40	5	323.9	6.0	214.4	4.4	8.1	3.7
			Total	20	303.4	26.4	204.3	21.0	5.7	35.1
		7	16	6	421.9	37.3	254.2	29.3	2.2	18.2
			24	3	485.6	2.5	292.3	2.1	4.1	12.2
			32	4	539.3	15.1	315.8	9.9	6.5	32.3
			40	3	471.4	6.8	293.7	6.1	8.1	6.2
			Total	16	472.5	23.2	284.1	18.5	4.7	55.3
		8	16	2	391.3	2.4	237.3	3.7	1.4	21.4
			24	5	355.1	20.4	221.4	16.9	2.6	11.5
			Total	7	365.4	16.9	225.9	14.1	2.3	26.1
	Gravel	4	24	3	316.1	19.3	214.4	20.5	13.7	2.2
		5	16	3	421.3	2.6	263.8	2.3	9.4	2.1
			24	3	403.1	35.4	270.5	27.0	11.0	10.9
			32	3	422.4	6.9	275.4	7.1	15.6	9.0
			40	3	433.3	8.4	279.6	1.1	17.5	2.9
			Total	12	420.1	15.5	272.3	12.1	13.4	26.1
		6	16	4	306.9	45.8	219.4	35.1	8.7	6.7
			24	5	380.2	34.4	262.4	28.9	10.2	10.8
			32	5	278.6	28.0	200.9	31.4	12.7	4.7
			40	5	343.2	2.7	240.4	4.5	16.9	5.3
			Total	19	328.3	30.6	231.4	26.5	12.3	26.8
		7	16	6	424.5	30.6	269.6	26.0	4.2	19.0
			24	7	384.8	29.9	249.9	22.7	6.2	12.9
			32	5	352.2	30.3	232.9	15.2	8.3	20.5
			40	3	381.9	2.7	253.6	3.0	11.2	2.7
			Total	21	388.0	27.3	252.0	20.4	6.8	38.2
		8	24	3	365.8	10.6	232.9	7.2	3.0	3.3
100	Limestone	6	8	3	126.0	2.8	90.8	8.4	1.2	10.5
			16	3	154.5	6.5	108.7	8.1	3.0	10.0
			24	3	208.2	11.9	142.4	11.8	5.4	2.8
			Total	9	162.9	23.7	114.0	21.8	3.2	56.3
	Gravel	6	8	3	169.4	5.2	124.4	7.7	4.1	3.1
			16	3	213.2	4.7	151.0	4.7	6.9	5.3
			24	3	219.1	4.7	167.7	2.9	9.8	6.3
			Total	9	200.6	12.5	147.7	13.5	7.0	35.8

TABLE 18. MEASURED DYNAMIC MODULUS FOR ASPHALT CONCRETE MIXTURES (REFS 31 AND 48)

Material Description	Load Frequency	Load Duration, sec	No. of Load Repetitions	Temp., °F	Dynamic Modulus, psi
California type B; 1/2 in. max. med. aggr.; 85-100 pen. asphalt	30 cpm	0.1	100	70	300,000
				90	70,000
Georgia std. A; 1-1/2 in. max. aggr.; 85-100 pen. asphalt	20 cpm	0.1	10,000	72	220,000
				89	100,000
California type B; 3/8 in. max. med. aggr.; 85-100 pen. asphalt	30 cpm	0.1	100	40	2,500,000
				55	1,500,000
				70	500,000
				100	50,000
Asphalt Institute mix IVb; 1/2 in. max. aggr.; 60-70 and 85-100 pen. asphalts	1 cps	—	100-300	40	580,000-2,250,000
	1 cps			70	82,000-710,000
	1 cps			100	25,000-155,000
	4 cps			40	830,000-2,700,000
	4 cps			70	176,000-1,200,000
	4 cps			100	45,000-260,000
	16 cps			40	1,100,000-3,400,000
	16 cps			70	250,000-1,600,000
	16 cps			100	75,000-450,000

Factors Affecting Repeated-Load Resilient Moduli

Prior to analyzing the effects of the various factors, an analysis of variance was conducted, with stress level, asphalt content, and aggregate type as the main factors. Temperature was not included since only one asphalt content was considered for specimens tested at 50 and 100°F, and different stress levels were used at the different temperatures in order to obtain reasonable values of stress-strength ratios and fatigue lives.

The results of the analysis of variance are summarized in Table 19. The effect of asphalt content was very significant (α = 0.05%) and it can be seen that the linear effect of asphalt content was less significant (α = 10%) than its quadratic effect (α = 0.05%). Stress level was shown to be relatively insignificant while the interaction effect between asphalt content and aggregate type was significant at an α-level of 0.5%. Thus, from the analysis of variance, asphalt content (quadratic effect) and the interaction between asphalt content and aggregate type had significant effects on the total resilient modulus of asphalt mixtures.

The effects of the above factors and temperature are discussed in more detail in the following sections.

Stress level. The effect of stress level on the values of both the instantaneous and the total resilient moduli was inconsistent and shown by the analysis of variance not to be significant. There appeared to be an increase in the resilient modulus with increased stress level for tests conducted at 100°F; this trend, however, was not observed for tests conducted at 50 and 75°F, as indicated in Figs 76 and 77.

Schmidt (Ref 69) found that the resilient modulus decreased with increased flexural stress and decreased slightly with increased direct tension or direct compression stress. Other studies utilizing the triaxial test (Refs 15, 74, and 77) indicated that the resilient modulus increased with an increase in confining pressure or a decrease in deviator stress. However, Schmidt (Ref 69) also presented data indicating that the magnitude of the indirect tensile stress had a very slight effect on the resilient modulus. In addition, Howeedy and Herrin (Ref 24), utilizing a constant displacement repeated compressive stress test on cold mixes, found that there was no regular relationship between the resilient modulus and stress.

TABLE 19. LEAST SQUARES ANALYSIS OF VARIANCE FOR REPEATED-LOAD TOTAL RESILIENT MODULUS: 75°F

Source of Variation	Degree of Freedom		Mean Squares x 10^8	F Values	Significance Level*, %	
Aggregate type	1		39.4	1.846	NS	
Asphalt content	2		214.5	10.056	0.05	
linear		1	74.3	3.484		10
quadratic		1	354.7	16.628		0.05
Stress level	3		13.4	.627	NS	
linear		1	28.7	1.346		NS
quadratic		1	0.9	.041		NS
cubic		1	10.6	.495		NS
Aggr. x Asph.	2		141.3	6.624	0.5	
Aggr. x Stress	3		17.2	.805	NS	
Asph. x Stress	6		19.4	.911	NS	
Remainder	81		21.3			
Total	98					

*NS - not significant at $\alpha \leq 10\%$

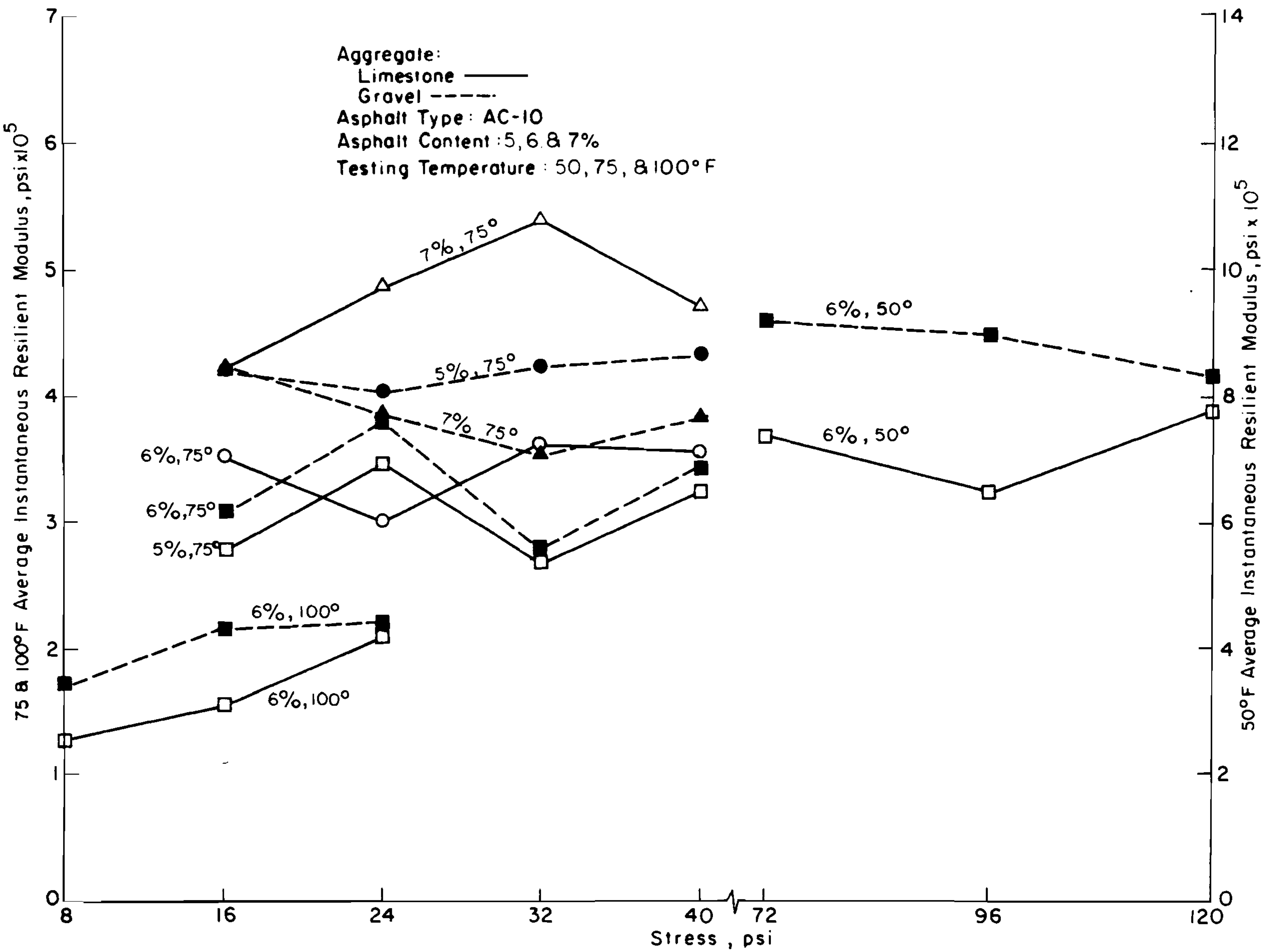


Fig 76. Effects of stress level and temperature on average instantaneous resilient modulus.

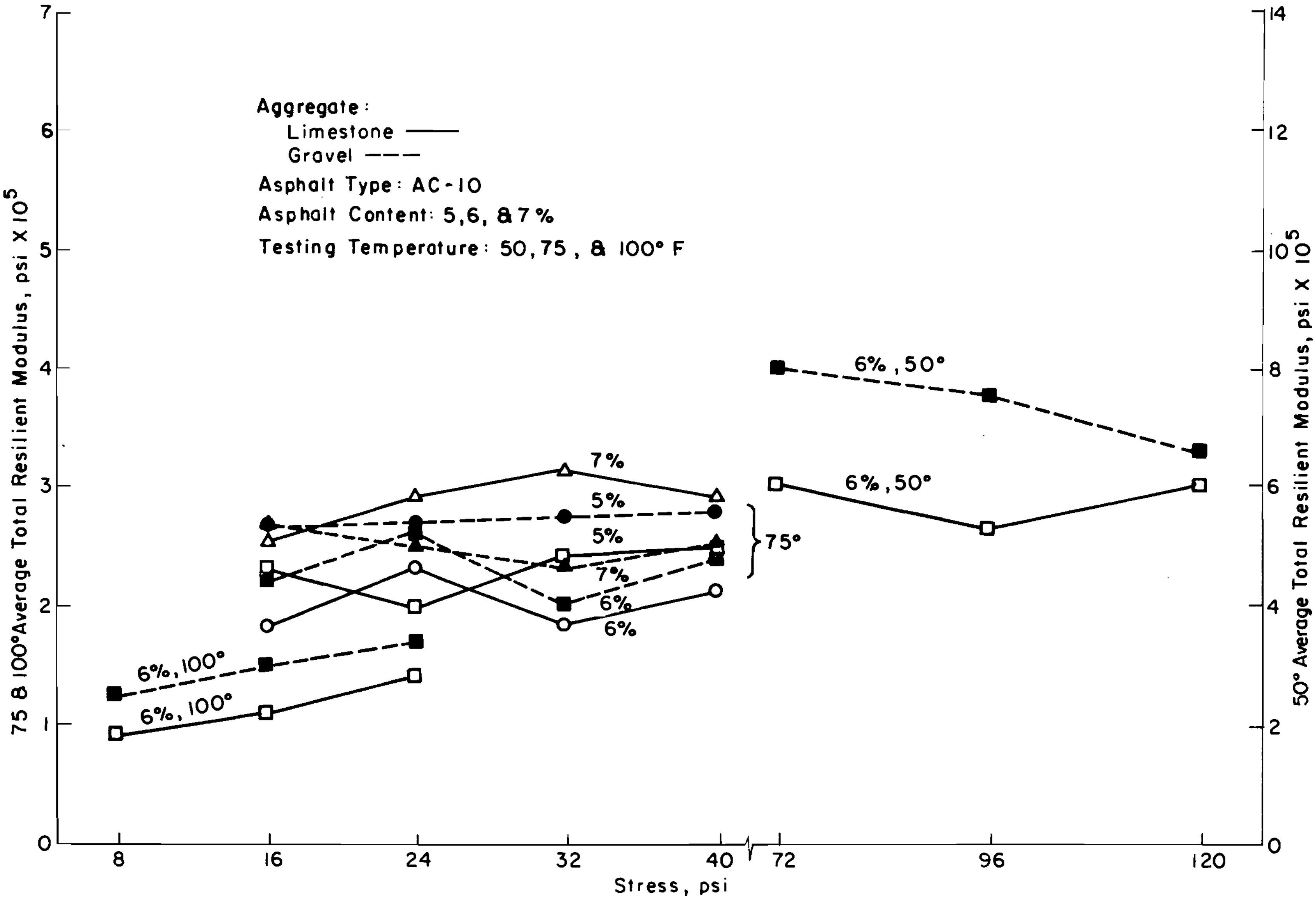


Fig 77. Effect of stress level and temperature on average total resilient modulus.

Thus, one may conclude from the results of this study that the effect of stress on the resilient modulus is either insignificant or that there is probably a complex interaction between stress and other factors, such as asphalt content, aggregate, and testing temperature, which causes the main effect of stress to be unimportant, although no significant interaction effects were detected by the analysis of variance.

Asphalt content. The effect of asphalt content on dynamic modulus is not definite as yet. Shook and Kallas (Ref 71) have found that

> for constant compactive efforts, which result in air voids between 1 and 12 percent, the modulus may increase, increase and then decrease, or decrease as the asphalt content is increased from 4 to 6 percent, depending on temperature and the level of air voids or VMA.

Schmidt (Ref 69) reported an optimum asphalt content for maximum resilient modulus, although the optimum occurred at a plateau. However, the study was too limited to allow any broad conclusions. In this study, the data seemed to indicate that the maximum dynamic modulus occurred at an asphalt content of 5 percent for the gravel mixtures and of 7 percent for the limestone mixtures (Figs 78 through 80); however, an optimum was not apparent or well defined.

Testing Temperature. Due to the doubtful effects of stress and asphalt content on resilient modulus, the effect of temperature was evaluated by comparing values for different temperatures at the same stress level and asphalt content. The results at 16 and 24 psi for mixtures containing 6 percent asphalt can be compared in Figs 76 and 77, which indicate that the resilient modulus was smaller at 100°F than at 75°F and thus probably decreases with increased testing temperatures. A wider range of levels of factors is, however, essential for any definite conclusions. In order to indicate the likely effect of 50°F, values for mixtures containing 6 percent asphalt were averaged over the range of stress levels considered at each temperature (72 to 120 psi for 50°F, 16 to 40 psi for 75°F, and 8 to 24 psi for 100°F). Figure 81 illustrates the effect of testing temperature on the resilient modulus, from which it is evident that a decrease in testing temperature produced an increase in the repeated-load resilient modulus. The effect is more significant in the lower temperature range, which was also true for static modulus of elasticity (Chapter 4).

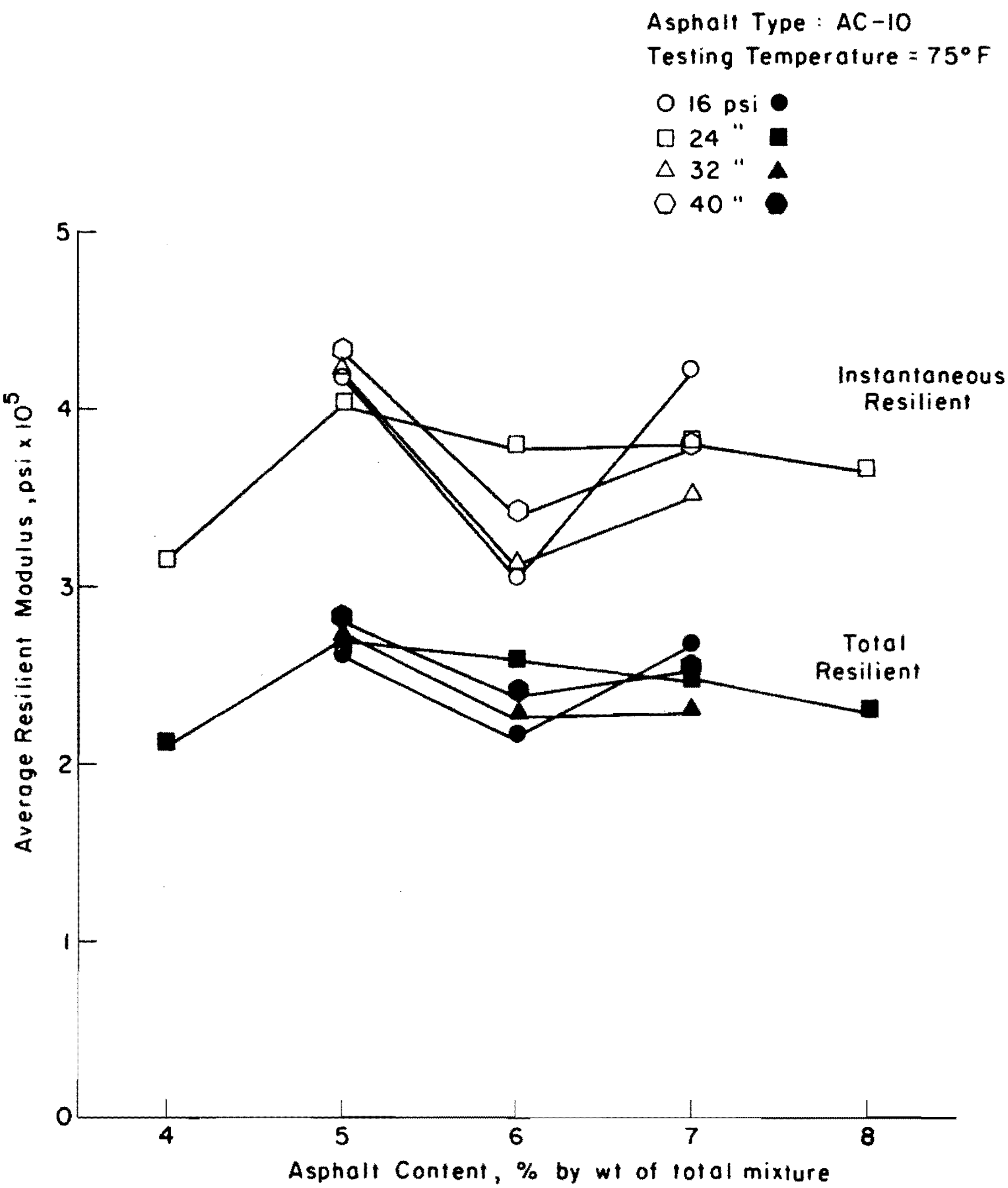


Fig 78. Relationships between average instantaneous and total resilient modulus, and asphalt content for gravel mixtures.

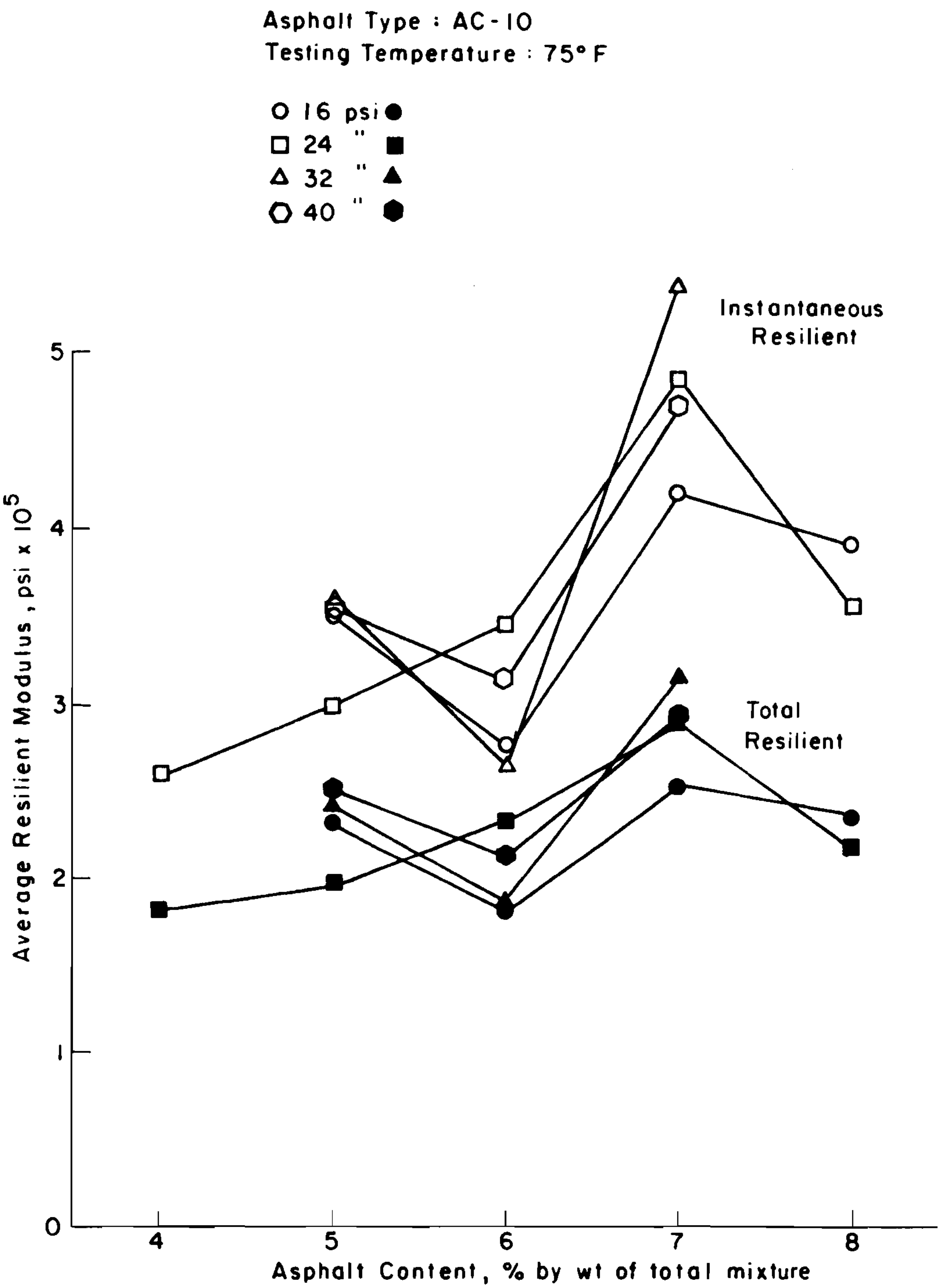


Fig 79. Relationship between average instantaneous and total resilient modulus, and asphalt content for limestone mixtures.

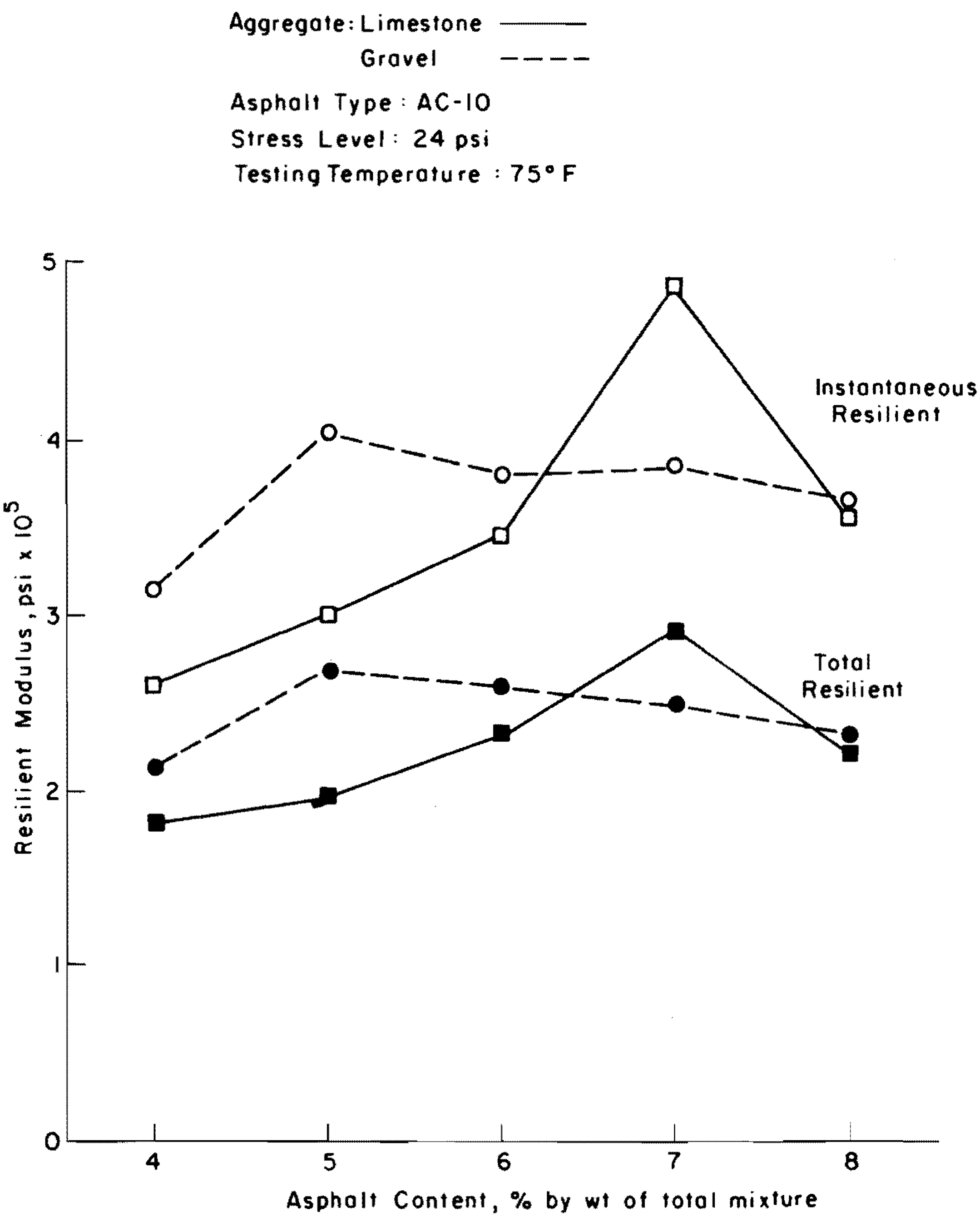


Fig 80. Effect of aggregate type and asphalt content on average instantaneous and total resilient modulus for limestone and gravel mixtures.

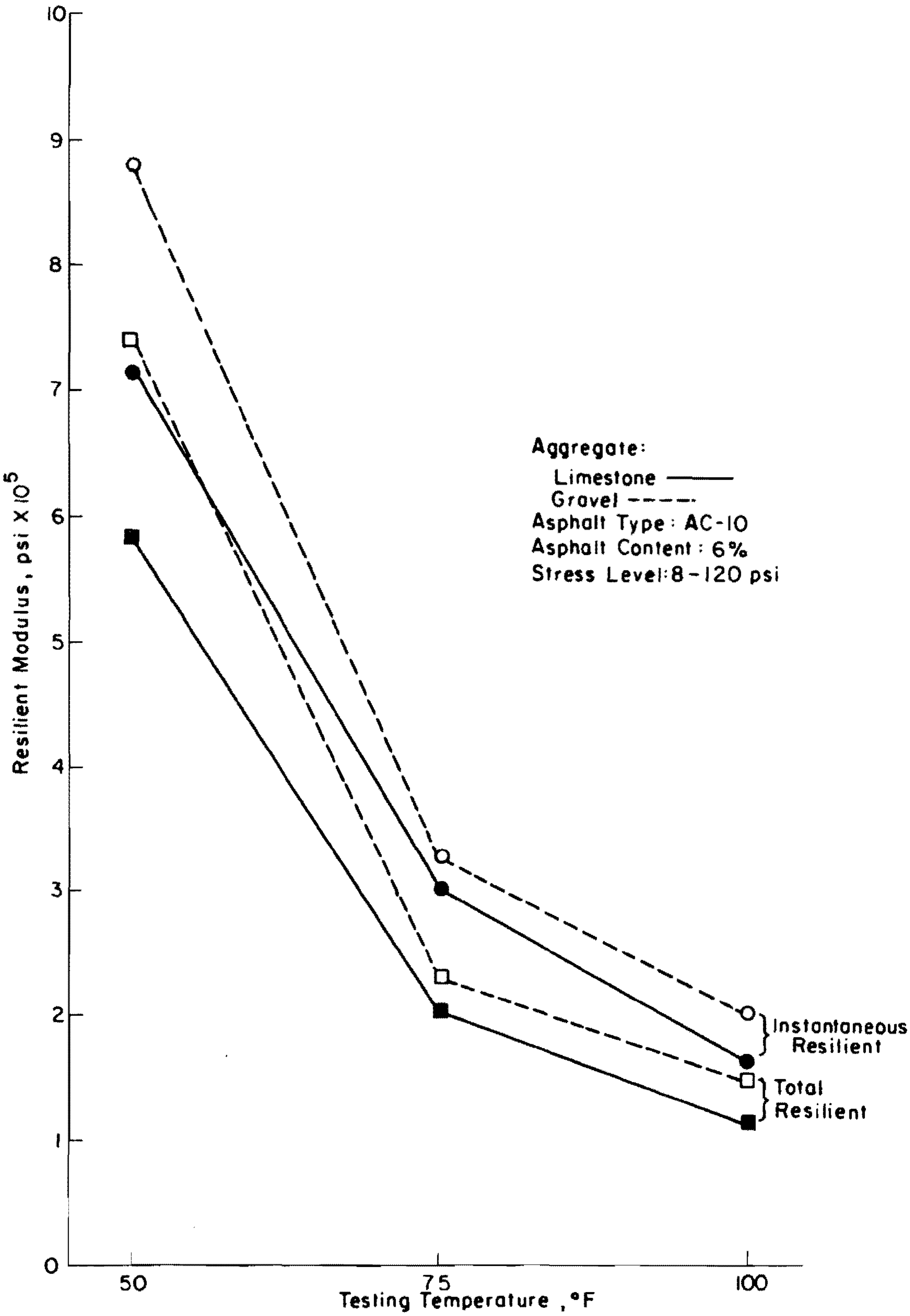


Fig 81. Effect of testing temperature on average repeated-load resilient modulus.

Aggregate Type. The effect of aggregate type on the repeated-load resilient modulus is shown in Figs 76, 77, 80, and 81. Fig 80 indicates that the resilient moduli of the gravel mixtures were larger than the resilient moduli of the limestone mixtures except for mixtures containing 7 percent asphalt and tested at 75°F. The analysis of variance, however, indicates that the difference is not significant.

Effect of Assumed Constant Poisson's Ratio on the Resilient Modulus

There seems to be a marked effect of repeated loads on the values of Poisson's ratio, as will be presented later in the chapter. However, since a constant value of Poisson's ratio can be and has been assumed in the analysis of dynamic indirect tension results, the effect of this on the values of the total resilient modulus was evaluated by assuming values of 0.15, 0.25, and 0.35. These values were selected since many studies have found them to be reasonable for asphalt mixtures. Figure 82 compares total resilient moduli using calculated and assumed Poisson's ratio values for a gravel specimen containing 6 percent asphalt and tested at a stress level of 24 psi. This figure indicates that for a constant Poisson's ratio the moduli were larger during the earlier loading period but that the moduli decreased more rapidly with an increased number of stress applications. The results seem to indicate that a more stable modulus is obtained by taking into account the changes in Poisson's ratio as a result of repeated-load applications.

Modulus of Cumulative Total Deformation

The effect of repeated loads on the modulus of cumulative total deformation estimated as explained earlier, in Chapter 3, was investigated. Since this is based on the cumulative total deformation it is felt that it might reflect the loading history of the specimen and thus be a measure of the total performance from the start of load application up to the instance in question. This is different than for the resilient modulus in that the resilient modulus, a measure of the specimen's elasticity, represents the ability of the mixture to resist resilient deformations at any instance, without reflecting the history of the specimen.

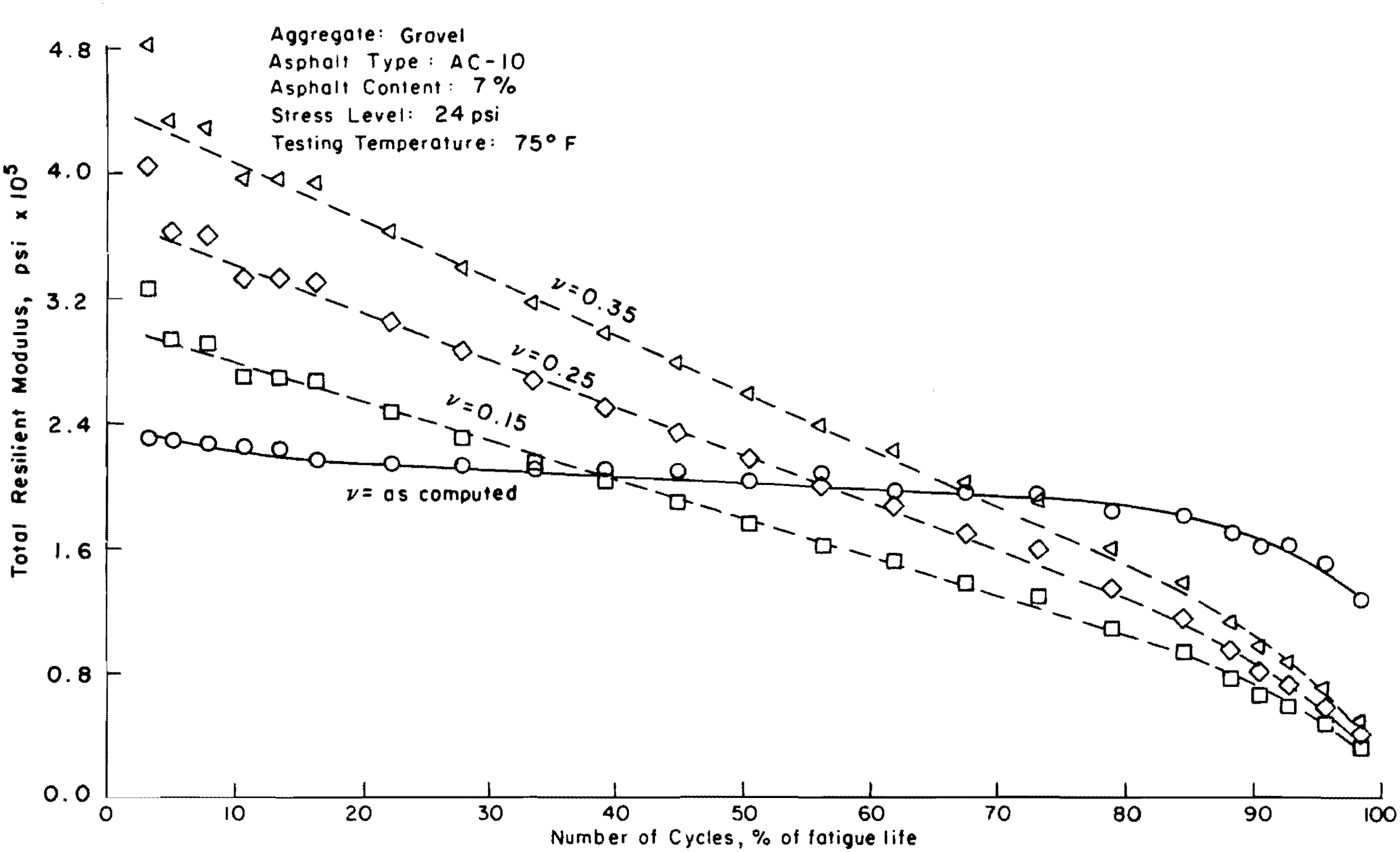


Fig 82. Effect of constant Poisson's ratio on the changes in total resilient modulus with repeated-load applications.

Figure 83 is a typical representation of the variation in the modulus of cumulative total deformation, with the number of load repetitions expressed as a percent of fatigue life. The general shape is similar to that obtained in an earlier study by Deacon (Ref 6) for the stiffness of an asphalt mixture subjected to repeated flexure: an initial rapid drop in modulus value followed by a prolonged gradual decrease and a final rapid drop at failure.

Unlike the resilient modulus, which can be used in the elastic layered theory or the beam theory, the actual practical significance of the modulus of cumulative total deformation is yet to be established. Because of this no detailed analysis was carried out at this stage. However, Table 17 contains mean values (averages of values at 30, 40, 50, 60, and 70 percent of fracture life) which ranged between 1,200 psi and 76,600 psi, depending on asphalt content, stress level, testing temperature, and aggregate type. The values are generally low because of the large deformations involved.

The effects of stress level and asphalt content on the modulus values are shown in Figs 84 and 85 respectively. The figures seem to indicate that slightly steeper slopes and higher modulus values are obtained for higher stress levels and lower asphalt contents. These observations are also evident from data presented in Table 17. The effect of temperature on the modulus of cumulative total deformation is similar to that on the repeated-load resilient modulus, with higher temperatures producing lower modulus values. Mixtures containing gravel aggregates had higher modulus values than those containing limestone aggregates.

Comparison of Static and Dynamic Moduli

Table 20 shows a comparison of modulus values estimated by static and repeated-load test methods. In the case of the repeated-load test, values presented in Table 20 were obtained by averaging the values shown in Table 17 over the range of stress levels considered. The coefficients of variation obtained by averaging over a range of stress levels did not differ significantly from those obtained for individual stress levels except in the case of the modulus of cumulative total deformation.

As can be seen in Table 20, the static modulus values are generally lower than the repeated-load modulus values (instantaneous and total resilient). This difference may be due to, among other possible reasons,

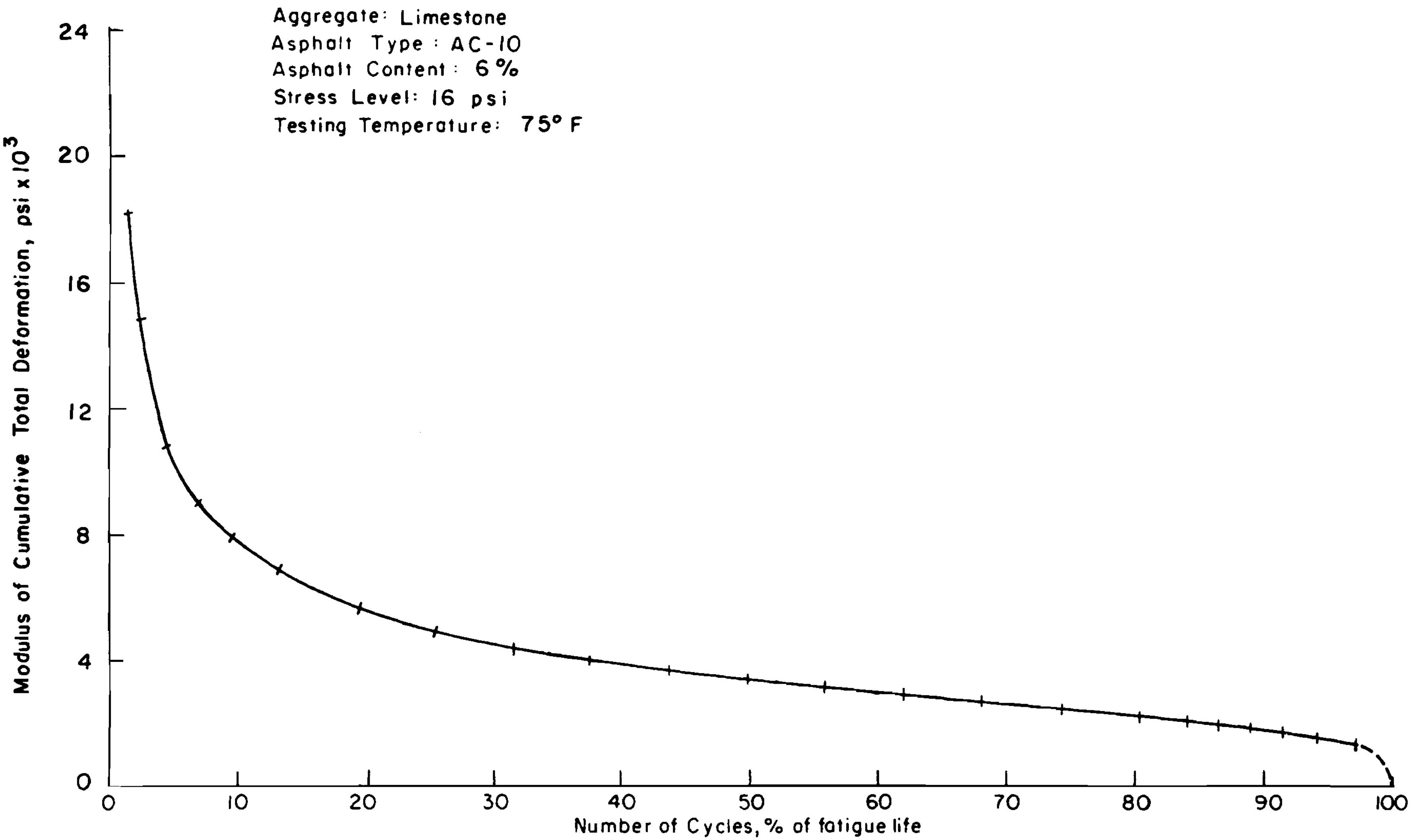


Fig 83. Effect of repeated loads on the modulus of cumulative total deformation.

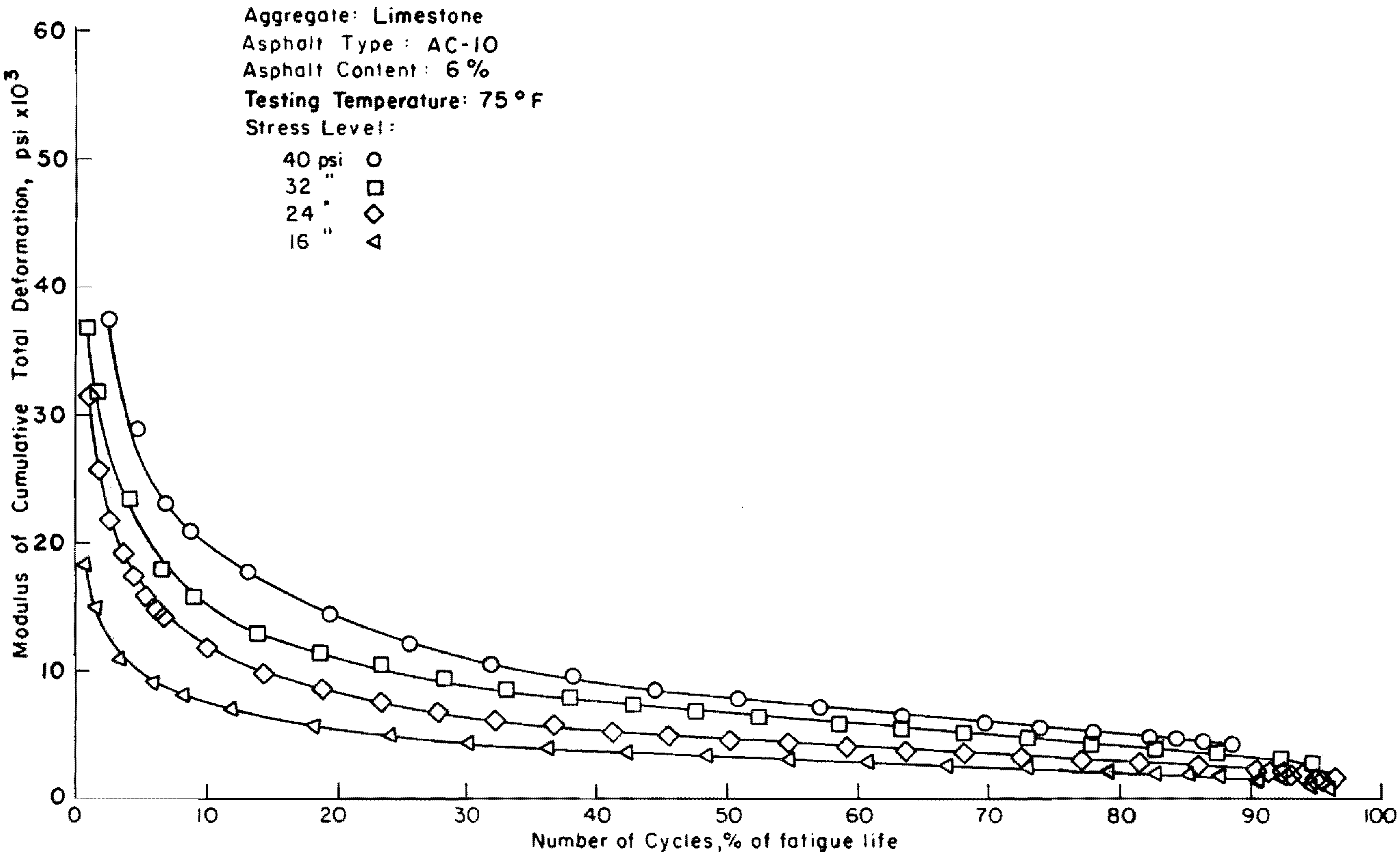


Fig 84. Effect of stress on the modulus of cumulative total deformation.

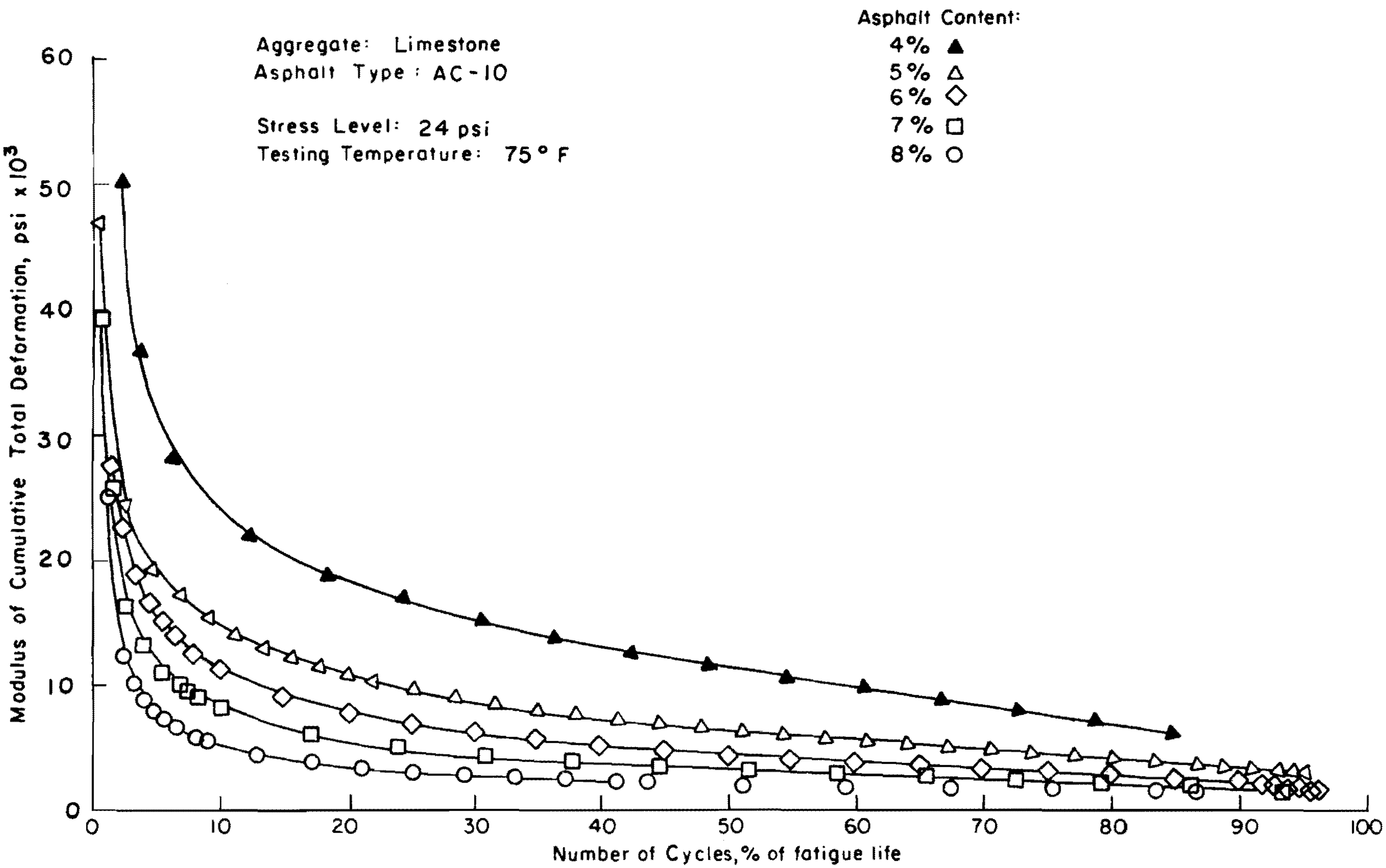


Fig 85. Effect of asphalt content on the modulus of cumulative total deformation.

TABLE 20. COMPARISON OF VARIOUS TYPES OF ESTIMATED MODULUS

Temperature, °F	Aggregate	Asphalt Content, %	Static Modulus E_S, psi x 10^3	Total Resilient Modulus E_{RT}, psi x 10^3	Instantaneous Resilient Modulus E_{RI}, psi x 10^3	Modulus of Cum. Total Def. E_{TT}, psi x 10^3	E_{RT}/E_{RI},* %	E_S/E_{RI},** %	E_{TT}/E_{RI},* %
50	Limestone	4	243.2						
		5	363.3						
		6	595.2	583.8	717.7	37.0	81	83	5.2
		7	489.7						
		8	292.0						
	Gravel	4	167.7						
		5	314.9						
		6	489.3	741.2	880.0	58.8	84	56	6.8
		7	652.1						
		8	549.7						
75	Limestone	4	71.1	183.2	260.1	14.6	70	27	5.7
		5	99.2	231.4	342.3	11.1	68	29	3.3
		6	114.6	204.3	303.4	5.7	66	38	2.3
		7	82.9	284.1	472.5	4.7	61	18	1.0
		8	56.5	225.9	365.4	2.3	62	15	0.6
	Gravel	4	73.6	214.4	316.1	13.7	68	23	4.5
		5	104.8	272.3	420.1	13.4	65	25	3.2
		6	156.0	231.4	328.3	12.3	71	48	4.0
		7	160.8	252.0	388.0	6.8	66	41	1.9
		8	105.4	232.9	365.8	3.0	64	29	0.8
100	Limestone	4	26.2						
		5	34.3						
		6	33.6	114.0	162.9	3.2	70	21	1.9
		7	16.1						
		8	5.0						
	Gravel	4	29.1						
		5	30.5						
		6	48.9	147.7	200.6	7.0	74	24	3.4
		7	34.3						
		8	18.0						

* Average of the ratio E_{RT}/E_{RI} or E_{TT}/E_{RI} for all specimens.

** Estimated by dividing average E_S by average E_{RI}.

the difference in load duration and rate of strain during testing. Whereas the static test lasted for about 3 to 5 seconds, the load duration in the repeated-load test was only 0.4 seconds.

At 75 and 100°F, the ratio of the repeated-load total resilient modulus E_{RT} to the repeated-load instantaneous resilient modulus E_{RI} ranges between 61 and 74 percent while the ratio of the static modulus of elasticity E_S to the repeated-load instantaneous resilient modulus is lower, ranging between 15 and 48 percent. At 50°F, these ratios are considerably higher, with values over 80 percent except for the ratio E_S/E_{RI} for the gravel mixtures, which is 56 percent. In the case of the modulus of cumulative total deformation E_{TT}, the ratio E_{TT}/E_{RI} is generally less than approximately 6 percent.

POISSON'S RATIO

Figure 86 shows what can be considered a typical shape of the curve of total resilient Poisson's ratio versus number of cycles. As indicated by the figure, there seems to be a gradual increase in Poisson's ratio with an increasing number of load applications until, at about 70 to 80 percent of the fracture life, the increase in Poisson's ratio is very rapid. This instance corresponds approximately to the instance at which both resilient and permanent strains increase rapidly and also at which there is a sharp decrease in resilient modulus.

It is noticed that Poisson's ratio exceeds the value 0.5 after several load applications. This contradicts the theoretical deduction that Poisson's ratio must be less than or equal to 0.5. In an attempt to explain this deviation from theory, attention is drawn to the derivation of the limitation for Poisson's ratio, which is presented in Appendix H. Two important assumptions made in the derivation of the limit are

(1) material is homogeneous, isotropic, and elastic, and
(2) volume change in tension is positive (expansion) while volume change in compression is negative (contraction).

For asphalt concrete specimens subjected to indirect tension loading; assumption 1 above is violated because the material is nonhomogeneous, anisotropic, and viscoelastic. Also, when a vertical crack occurs (internally or externally) from splitting tensile stresses, there is a tendency for the specimen to be pulled apart in the horizontal direction, with loose particles

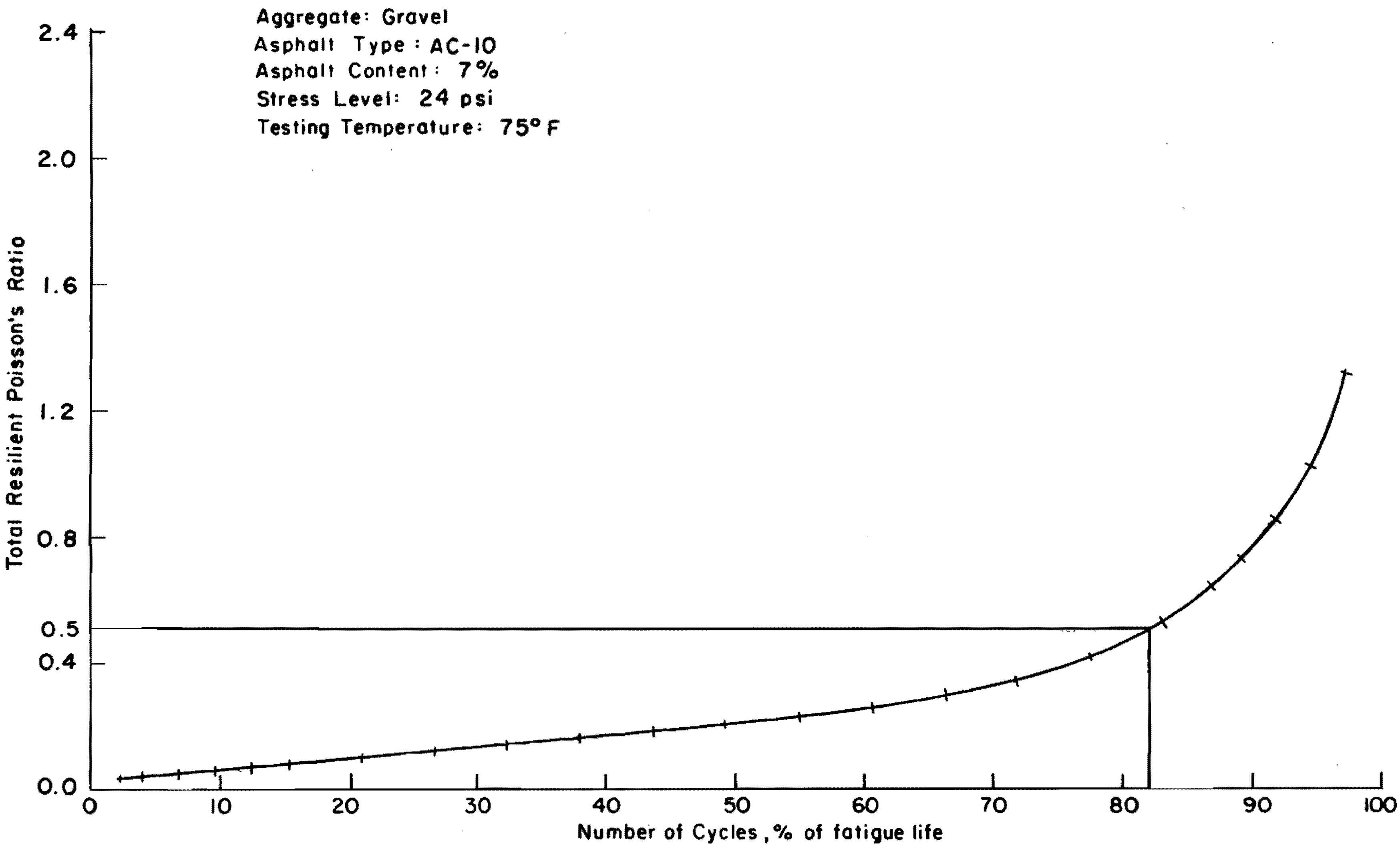


Fig 86. Effect of repeated loads on total resilient Poisson's ratio.

and air filling the created space, resulting in a net dilation rather than a contraction of the specimen, probably after sufficient load applications to cause initiation of cracks. Thus, assumption 2 above is also violated and the condition (as presented in Appendix H)

$$\Delta V = V_o \varepsilon \ (2\nu - 1) \geq 0$$

for the case of compression no longer holds. Instead one might write

$$\Delta V \lesseqgtr 0$$

for compression or possible dilation; that is

$$2\nu - 1 \lesseqgtr 0,$$

since

$$V_o > 0 \text{ and}$$
$$\varepsilon > 0.$$

Thus, $\nu \lesseqgtr 0.5$, indicating that Poisson's ratio can be greater than 0.5.

An attempt was made to relate the instant at which Poisson's ratio exceeds 0.5 with the variations in other properties, such as resilient and permanent strains. There was, however, no such relationship since the point at which Poisson's ratio exceeds 0.5 varied from specimen to specimen and ranged anywhere between 0 and 100 percent of fracture life. It thus seems that the point at which Poisson's ratio exceeds 0.5 must be a function of the initial physical condition of the specimen in addition to the number of load applications. Therefore, at this stage of research, it may suffice to suggest that the effect of repeated indirect tensile stresses is to increase Poisson's ratio, even beyond the limit 0.5, and that the point at which Poisson's ratio exceeds the limit 0.5 is a function of the initial physical conditions of the specimen as well as the magnitude of the repeated load and the environmental conditions.

Table 21 shows values of Poisson's ratio estimated by averaging values at 30, 40, 50, 60, and 70 percent of the fracture life of the specimens. These values ranged between negative values and values greater than 0.5.

SERVICE LIFE ESTIMATION

A study of this nature, in which the properties of each specimen of the asphalt mixtures have been extensively investigated, should provide a means of defining the service life of laboratory specimens subjected to repeated tensile stresses. It was felt that the combined variations of resilient strain, permanent strain, resilient modulus, and, possibly, Poisson's ratio should be a basis of estimating the specimen's service life. Figure 87 shows a superimposition of the total resilient and permanent strains, and the total resilient modulus for a single specimen of the mixture containing limestone aggregates and 5 percent asphalt and tested at a stress level of 24 psi.

As indicated by the figure, the specimen suffers excessive resilient and permanent strains and then a sharp reduction in the resilient modulus. Thus, as the specimen is subjected to repeated load applications, there are increasing permanent and resilient strains with an associated decrease in the resilient modulus. This continues until at about 70 to 80 percent of the eventual fracture life, at which time the specimen has cracked sufficiently, causing a sharp increase in permanent strain and resilient strain with the sharp decrease in the resilient modulus. This is probably the point at which the material stops rendering useful service without excessive maintenance in the case of field pavements. Thus, a point lying anywhere between 70 and 80 percent of the fracture life, probably closer to 80 percent, can be used to define the service life of the specimen.

TABLE 21. SUMMARY OF REPEATED-LOAD RESILIENT POISSON'S RATIO

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Specimens	Instantaneous Resilient Poisson's Ratio Mean	Instantaneous Resilient Poisson's Ratio CV, %	Total Resilient Poisson's Ratio Mean	Total Resilient Poisson's Ratio CV, %
50	Limestone	6	72	3	-0.10	93	-0.02	445
			96	3	-0.14	24	-0.07	61
			120	3	-0.11	52	-0.03	142
			Total	9	-0.12	52	-0.04	220
	Gravel	6	72	3	-0.10	31	-0.05	81
			96	3	-0.07	144	0.01	693
			120	3	-0.02	227	0.06	10
			Total	9	-0.06	106	0.01	209
75	Limestone	4	24	3	0.51	26	0.60	20
		5	16	3	0.38	33	0.40	28
			24	3	0.38	22	0.44	25
			32	3	0.41	24	0.45	19
			40	3	0.55	6	0.59	9
			Total	12	0.43	25	0.47	23
		6	16	5	0.20	120	0.29	91
			24	5	0.29	60	0.38	51
			32	5	0.16	36	0.24	25
			40	5	0.19	28	0.29	23
			Total	20	0.21	71	0.30	54
		7	16	6	0.55	52	0.65	47
			24	3	0.80	11	0.88	10
			32	4	0.70	11	0.77	9
			40	3	0.58	1	0.60	3
			48	1	0.93	–	1.01	–
			56	1	0.93	–	0.93	–
			Total	18	0.67	31	0.74	29
		8	16	2	0.72	5	0.80	5
			24	5	0.56	35	0.64	27
			Total	7	0.60	29	0.69	24
	Gravel	4	24	3	0.59	38	0.67	37
		5	16	3	0.40	15	0.39	19
			24	3	0.40	73	0.42	59
			32	3	0.31	10	0.39	5
			40	3	0.40	33	0.47	23
			Total	12	0.38	38	0.42	29
		6	16	4	0.00	–	0.04	396
			24	5	0.11	69	0.16	50
			32	4	0.04	100	0.13	23
			40	5	0.08	53	0.16	31
			Total	18	0.06	140	0.13	72
		7	16	6	0.25	71	0.35	62
			24	7	0.35	82	0.42	66
			32	5	0.21	124	0.29	96
			40	3	0.17	58	0.23	43
			Total	21	0.26	86	0.34	69
		8	16	1	0.70	–	0.79	–
			24	3	0.51	27	0.65	26
			Total	4	0.56	26	0.69	23
100	Limestone	6	8	3	0.55	9	0.71	13
			16	3	0.61	14	0.74	14
			24	3	0.51	6	0.59	12
			Total	9	0.56	12	0.68	15
	Gravel	6	8	3	0.35	10	0.43	5
			16	3	0.49	12	0.54	10
			24	3	0.34	35	0.46	36
			Total	9	0.39	25	0.48	21

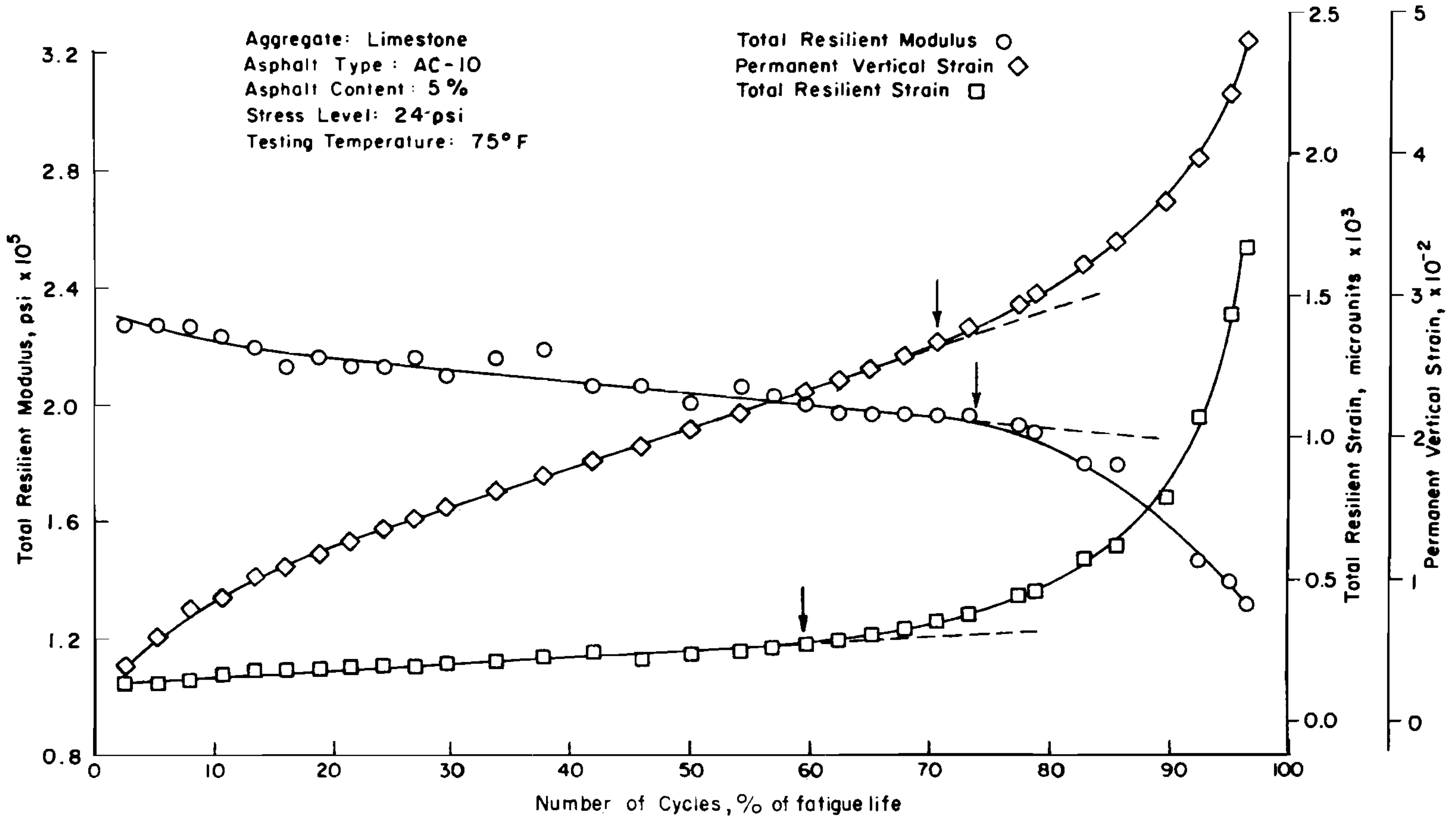


Fig 87. Superimposition of total resilient and permanent strains, and total resilient modulus.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

Within the limits of load, mixture and specimen, and temperature variables considered in this study, the following conclusions and recommendations were made.

CONCLUSIONS

General

The indirect tensile test is suitable for the study of fatigue and repeated-load characteristics of asphalt mixtures. This is important in view of its simplicity and the ease of conducting it.

Static Characteristics

(1) The optimum asphalt content for maximum tensile strength and static modulus of elasticity was less than the optimum for maximum bulk density.

(2) The effect of asphalt content on indirect tensile strength and modulus of elasticity decreased with an increase in temperature. Thus, at the higher temperatures the effect of asphalt content was less important and, therefore, the selection of an optimum asphalt content was less critical.

(3) Poisson's ratio varied very widely, ranging between negative values at 50°F and values near or greater than 0.5 at 100°F. There was no consistent relationship between asphalt content and Poisson's ratio.

(4) The indirect tensile strength of the asphalt mixtures could be estimated on the basis of asphalt content, aggregate type, and testing temperature as follows:

$$\log S_T = 0.3202 + 0.9380\,A - 0.0116\,T - 0.0577\,A^2 - 0.0899\,GA + 0.0148\,GA^2 - 0.0003\,TA^2 \qquad (4.1)$$

where

S_T = predicted indirect tensile strength, psi,

A = asphalt content, percent by weight of total mixture,

T = testing temperature, °F, and

G = coded value for aggregate (limestone = 0, gravel = 1).

(5) There was a relationship between the indirect tensile strength and static modulus of elasticity, which can be expressed as

$$\log E_S = 2.94 + 1.03 \log S_T \tag{4.3}$$

where

E_S = static modulus of elasticity, psi, and

S_T = indirect tensile strength, psi.

Additional tests should be conducted, including tests on other materials and for other conditions, in order to improve the relationship and extend its inference.

(6) There were no strong correlations between the following pairs of properties:

(a) indirect tensile strength and bulk density,

(b) modulus of elasticity and bulk density,

(c) indirect tensile strength and air void content, and

(d) modulus of elasticity and air void content.

Fatigue Characteristics

(1) A linear relationship was found to exist between the logarithm of fatigue life and

(a) the logarithm of tensile stress,

(b) the logarithm of stress difference, and

(c) the logarithm of initial strain defined as the applied repeated tensile stress divided by the average static modulus of elasticity.

These equations could be expressed in the following form:

$$N_f = K_2 \left(\frac{1}{\sigma}\right)^{n_2} \tag{2.5}$$

$$N_f = K_2' \left(\frac{1}{\Delta\sigma}\right)^{n_2} \tag{2.6}$$

$$N_f = K_1 \left(\frac{1}{\varepsilon_{mix}}\right)^{n_1} \tag{2.7}$$

where

N_f = fatigue life,

σ = applied repeated stress, psi,

$\Delta\sigma$ = stress difference, psi,

ε_{mix} = initial strain in mixture, and

K_1, n_1, K_2, n_2, and K_2' are constants.

(2) For the constants K_1, n_1, K_2, n_2, and K_2',

(a) K_1 ranged between 5.65×10^{-17} and 5.01×10^{-7}, n_1 ranged between 2.66 and 5.19, K_2 ranged between 3.26×10^{5} and 1.90×10^{13}, n_2 ranged between 2.66 and 5.19, and K_2' ranged between 1.41×10^{7} and 2.53×10^{16}; and

(b) there were linear relationships between n_1 and the logarithm of K_1, and between n_2 and the logarithm of K_2 (or K_2') as follows:

(i) with n_1 and n_2 as dependent variables,

$$n_1 = 0.938 - 0.261 \log K_1 \tag{5.7}$$

$$n_2 = 0.919 + 0.312 \log K_2 \tag{5.1}$$

$$n_2 = 0.734 + 0.266 \log K_2' \tag{5.2}$$

(ii) with n_1 and n_2 as independent variables,

$$\log K_1 = 3.211 - 3.719\, n_1 \tag{5.8}$$

$$\log K_2 = 2.886\, n_2 - 1.867 \tag{5.3}$$

$$\log K_2' = 3.485\, n_2 - 1.859 \tag{5.4}$$

Such relationships can simplify the fatigue life predictive equations by reducing the constants associated with them.

(3) There was an optimum asphalt content for maximum fatigue life, which was approximately equal to the optimums for indirect tensile strength and static modulus of elasticity but slightly less than the optimum for density. The optimum for fatigue life decreased slightly with increasing stress level for the limestone mixtures; however, it was concluded that stress did not affect the optimum asphalt content.

(4) The effect of aggregate type on fatigue life was not important, but, for low stress levels such as those experienced in the field, aggregate type could be a significant variable.

(5) Fatigue life increased with decreasing testing temperature.

(6) While there were no universally applicable methods of estimating fatigue life on the basis of applied stress or stress difference, the fatigue life could be estimated on the basis of its relationship with initial strain and stress-strength ratio (the ratio of applied repeated stress to indirect tensile strength) using the following regression equations:

$$N_f = 9.38 \times 10^{-8}\left(\frac{1}{\varepsilon_{mix}}\right)^{2.76} \tag{5.13}$$

$$N_f = 1.97 \times 10^{7}\left(\frac{1}{SSR}\right)^{3.18} \tag{5.20}$$

where

SSR = stress-strength ratio, percent.

(7) Based on the results from this study and previous studies, it was concluded that there is no general realtionship between laboratory fatigue life and air void content. Although a relationship may exist for a given mixture, other mixture and construction variables are probably more important and the effect of air void content on fatigue life will depend on the process by which the air void content is varied.

(8) There was a linear relationship between the logarithm of fatigue life and bulk density, but the relationship was stress dependent.

Repeated-Load Characteristics

(1) Both instantaneous and total resilient tensile strains exhibited a slight linear increase with an increase in the number of repeated stresses up to about 70 percent of the fracture life, at which point a more rapid increase occurred for additional stress repetitions, until complete fracture occurred.

(2) The relationship between permanent horizontal and vertical strains, and number of stress applications, could be divided roughly into 3 zones: (1) a conditioning zone, represented by the first 10 percent of the fracture life in which a rapid increase in strain occurred; (2) a relatively stable zone, lying between about 10 and 70 percent of the fracture life, in which there was a gradual and linear increase of strain with additional repeated stresses; and (3) a failure zone, represented by the last 30 percent of fracture life in which there was a rapid increase in strain with additional stress applications.

(3) The relationships between instantaneous resilient modulus, total resilient modulus, or modulus of individual total deformation, and the number of stress applications, could also be divided into three zones. During the first 10 percent of the fracture life, the shape of the relationship was uncertain due to initial adjustment to load and possible additional compaction. However, between

about 10 and 80 percent, the moduli decreased linearly with increasing stress applications. Beyond about 80 percent the moduli decreased very sharply until complete fracture.

(4) The rate of deterioration or decay of total resilient modulus with stress applications, evaluated in terms of the slope of the approximately linear portion, ranged between 5 and 990 psi/cycle. For instantaneous resilient modulus the slopes ranged between 7 and 3,000 psi/cycle. For both instantaneous and total resilient moduli, the rate of moduli decay increased with increasing stress level and increasing testing temperature, and there was an optimum asphalt content for minimum rate of decay, which corresponded to the optimum for fatigue life.

(5) The effect of assuming that Poisson's ratio remains constant during the entire loading period for a specimen was to increase the initial modulus and the rate of decay of both instantaneous and total resilient moduli.

(6) Average values of instantaneous resilient modulus, estimated as the average of the values lying between 30 and 70 percent of fracture life, ranged between 126,000 and 918,000 psi, while similar values of total resilient modulus of elasticity ranged between 91,000 and 802,000 psi.

(7) The effects of stress level and asphalt content on average instantaneous and total resilient moduli are uncertain; however, there was an increase in the instantaneous and total resilient moduli with decreasing testing temperature.

(8) The relationship between modulus of cumulative total deformation and number of stress applications indicated an initial rapid drop in modulus, followed by a prolonged period of gradual decrease, and a final sharp drop in the failure zone.

(9) Average values of modulus of cumulative total deformation were generally low, ranging from 1,200 to 76,600 psi. These values increased with decreasing asphalt content, increasing stress level, and decreasing testing temperature.

(10) Average values of instantaneous and total resilient moduli were higher than average values of static modulus of elasticity and modulus of cumulative total deformation. The ratio E_S/E_{RI} ranged between 15 and 83 percent while E_{TT}/E_{RI} was generally less than about 6 percent.

(11) Poisson's ratio increased with increasing stress applications.

(12) The service life of a laboratory specimen subjected to indirect tensile fatigue loading was estimated to be approximately 80 percent, or somewhere between 70 and 80 percent of its fracture life.

RECOMMENDATIONS

Immediate

(1) The use of static and repeated-load indirect tensile tests is

recommended for estimating various properties of asphalt mixtures.

(2) If strength tests cannot be conducted, the following equation may be used to estimate the indirect tensile strength:

$$\log S_T = 0.3202 + 0.9380\ A - 0.0116\ T - 0.0577\ A^2 - 0.0899\ GA + 0.048\ GA^2 - 0.0003\ TA^2 \qquad (4.1)$$

where

S_T = predicted indirect tensile strength, psi,

A = testing temperature, °F, and

G = coded value for aggregate (limestone = 0, gravel = 1).

(3) If it is not possible or practical to measure deformations, the static modulus of elasticity can be estimated from the indirect tensile strength by using the equation

$$\log E_S = 2.94 + 1.03 \log S_T \qquad (4.3)$$

where

E_S = static modulus of elasticity, psi, and

S_T = indirect tensile strength, psi.

The availability of the above equation should, however, not be used as a justification for not making the necessary measurements and it should be recognized that errors can be expected from its use.

(4) Fatigue life of asphalt mixtures may be estimated by using either of the following equations:

(a) $$N_f = 9.38 \times 10^{-8}\left(\frac{1}{\varepsilon_{mix}}\right)^{2.76} \qquad (5.13)$$

(b) $$N_f = 1.97 \times 10^{7}\left(\frac{1}{SSR}\right)^{3.18} \qquad (5.20)$$

where

ε_{mix} = initial mixture strain, estimated as the ratio of repeated tensile stress to the static modulus of elasticity, and

SSR = stress-strength ratio, estimated as the ratio of repeated tensile stress to indirect tensile strength, percent.

Long Term

Future research on the tensile characterization of pavement materials should include

(1) additional study of the fatigue and repeated-load resilient properties of asphalt mixtures, considering a wider range of factors such as aggregate gradation and temperature, thus extending the inferences of the various relationships developed in this study;

(2) additional study on the fatigue and repeated-load resilient properties of portland cement concrete and cement-treated and lime-treated pavement materials;

(3) use of available static, fatigue, and repeated-load resilient properties of asphalt mixtures in the formulation of design procedures for asphalt mixtures and modification of design procedures for asphalt concrete pavements;

(4) further evaluation of the modulus of cumulative total deformation and its possible use in the design of asphalt mixtures with respect to permanent deformation and possible correlations with inservice pavement performance; and

(5) further research on the effects of repeated stresses on Poisson's ratio, with consideration given to the possibility of using repeated-load values in the elastic design procedures.

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APPENDIX A

SUMMARY OF STRAIN-FATIGUE LIFE AND STRESS-FATIGUE LIFE REGRESSION COEFFICIENTS (K_1, n_1, K_2, AND n_2) FOR VARIOUS MIXTURES AND TEST METHODS

TABLE A-1. SUMMARY OF STRAIN-FATIGUE LIFE AND STRESS-FATIGUE LIFE REGRESSION COEFFICIENTS FOR VARIOUS MIXTURES AND TEST METHODS

$$N_f = K_1 \left(\frac{1}{\epsilon_{mix}}\right)^{n_1}, \quad N_f = K_2 \left(\frac{1}{\sigma}\right)^{n_2}$$

Test Type and Reference	Mixture	Asphalt Type, pen.	Asphalt Content, %	Temperature, °F	K_1	n_1	Corr. Coef.	K_2	n_2	Corr. Coef.
Beam Flexure, Controlled-Stress (Ref 48)	British 594 (Granite)	40-50	7.9	68	6.11×10^{-6}	3.38	.79	1.36×10^{11}	2.87	.45
	California (Granite)									
	- Coarse	85-100	6.0	68	3.20×10^{-5}	2.49	.93	1.64×10^{11}	3.69	.92
	- Medium	85-100	6.0	68	2.87×10^{-6}	2.83	.90	2.11×10^{12}	4.04	.90
	- Fine	85-100	6.0	68	8.91×10^{-7}	2.95	.95	1.55×10^{11}	3.51	.95
	- Medium	60-70	6.0	68	1.12×10^{-7}	3.26	.95	7.29×10^{12}	4.21	.89
	- Medium	40-50	6.0	68	1.03×10^{-10}	4.01	.96	1.97×10^{15}	4.93	.89
	(Basalt)									
	- Medium	60-70	6.2	68	1.34×10^{-7}	3.22	.90	6.01×10^{10}	3.24	.87
	Gonzales (Granite)									
	- Field, Surf.	85-100	5.9	40	4.35×10^{-8}	3.37	.58	4.10×10^{18}	6.06	.86
	- Field, Base	85-100	4.6	40	1.01×10^{-12}	4.56	.79	4.64×10^{16}	5.72	.75
	- Lab, Base	85-100	4.7	68	6.54×10^{-12}	4.69	.83	1.37×10^{13}	3.86	.64
	(Shale)									
	- Field, Surf.	85-100	5.9	68	2.12×10^{-8}	3.60	.87	2.41×10^{13}	4.82	.90
	- Lab, Surf.	85-100	6.0	40	1.10×10^{-10}	4.17	.79	1.78×10^{16}	5.09	.79
	Morro Bay (Sandstone)									
	- Field, Surf.	85-100	5.9	40	7.08×10^{1}	0.70	.23	1.55×10^{16}	5.16	.73
	- Field, Surf.	85-100	5.9	68	1.60×10^{-4}	2.35	.59	1.47×10^{14}	5.07	.67
	- Lab, Surf.	85-100	5.9	40	4.87×10^{-1}	1.43	.30	5.98×10^{17}	5.22	.52
	- Lab, Surf.	85-100	5.9	68	7.51×10^{-6}	2.79	.89	8.99×10^{15}	5.34	.83
	Folsom (Igneous Quartzitic)									
	- Field, Surf.	85-100	4.9	68	3.58×10^{-6}	2.80	.84	5.59×10^{13}	4.51	.76
	- Lab, Surf. 1 & 2	85-100	4.9	40	5.20×10^{-11}	4.08	.95	1.16×10^{18}	5.71	.92
	- Lab, Surf. 1 & 2	85-100	4.9	68	2.81×10^{-8}	3.47	.91	3.29×10^{14}	5.72	.79
	- Field, Base	85-100	4.2	68	1.09×10^{0}	1.17	.37	8.00×10^{7}	1.85	.24
	- Lab, Surf. 3 & 4	85-100	4.9	40	8.35×10^{-12}	4.17	.92	1.55×10^{16}	4.97	.90
Beam Flexure, Controlled-Stress (Ref 32)	Colorado A.C.									
	- Surface	57	5.6	70	2.73×10^{-7}	3.25	.91	3.90×10^{18}	6.87	.95
	- Base	57	5.6	70	2.01×10^{-5}	2.69	.98	1.02×10^{15}	5.11	.99
	Colorado Sand Base									
	- L.S.	57	8.6	70	8.97×10^{-7}	3.25	.99	9.04×10^{14}	4.68	.99
	- H.S.	57	8.0	70	2.82×10^{-11}	4.60	—	7.07×10^{14}	4.71	—
	Ontario A.C. Surface	84	5.7	70	1.37×10^{-6}	3.27	.97	2.08×10^{15}	5.16	.99
	Wash. St. Univ. Test Track-Ring 2									
	- A.C.	90	5.2	70	6.52×10^{-5}	2.50	.98	7.53×10^{14}	5.21	.98
	- A.T.B.	59	3.0	70	2.52×10^{-9}	3.58	.93	7.88×10^{15}	5.93	.94
	Lab. Study A.C.	84	6.0	55	2.32×10^{-9}	3.99	.92	8.51×10^{20}	6.89	.94
				70	4.00×10^{-6}	3.08	.97	6.52×10^{17}	6.17	1.00
				85	1.40×10^{-6}	3.45	.96	1.91×10^{15}	5.50	.98
Rotating Cantilever Bending, Controlled-Stress (Ref 60)*	HRA (base course) - A1	40-50	6.0	50	3.2×10^{-15}	5.1		3.7×10^{16}	5.4	
	- A2	40-50	6.0	50	1.3×10^{-13}	4.6		7.2×10^{14}	4.7	
	- A3	40-50	6.3	50	1.3×10^{-10}	3.9		1.2×10^{14}	4.5	
	- A4	40-50	6.0	50	2.5×10^{-13}	4.6		2.1×10^{16}	5.4	
	- A5	40-50	6.0	50	4.0×10^{-17}	5.5		2.1×10^{14}	4.5	
	- A6	40-50	6.0	50	1.3×10^{-13}	4.5		7.3×10^{12}	4.1	
	- A7	40-50	6.3	50	7.9×10^{-12}	4.2		1.0×10^{16}	5.3	
	- A8	30-40	6.0	50	1.6×10^{-14}	4.7		2.0×10^{15}	4.8	
	- A12	40-50	5.7	50	3.2×10^{-8}	3.2		1.2×10^{14}	4.4	
	- A13	40-50	6.8	50	1.0×10^{-7}	3.1		1.1×10^{12}	3.5	
	- A14	40-50	5.6	50	1.0×10^{-15}	5.2		5.1×10^{16}	5.3	
	- G	40-50	6.0	varies	8.8×10^{-15}	5.0		8.5×10^{17}	5.9	

*K_2 converted from those associated with stress in N/mm^2 to those associated with stress in psi.

(Continued)

TABLE A-1. (Continued)

Test Type and Reference	Mixture	Asphalt Type, pen.	Asphalt Content, %	Temperature, °F	K_1	n_1	Corr. Coef.	K_2	n_2	Corr. Coef.
Rotating Cantilever Bending, Controlled-Stress (Ref 60)*	HRA (wearing course)									
	- B9	40-50	7.1	50	2.5×10^{-14}	4.9		6.2×10^{19}	6.3	
	- B10	40-50	7.9	50	1.3×10^{-14}	5.1		1.0×10^{17}	5.1	
	- D5	40-60	7.7	50	1.6×10^{-15}	5.3		1.5×10^{17}	5.3	
	- D6	30-40	7.7	50	2.5×10^{-17}	5.8		1.0×10^{19}	5.8	
	- D7	Blown 40-60	7.7	50	1.6×10^{-12}	4.5		2.0×10^{17}	5.5	
	DBM (base course) - B1	190-210	4.3	50	2.5×10^{-12}	4.0		1.2×10^{12}	4.2	
	- B2	90-110	5.0	50	1.5×10^{-10}	3.8		8.9×10^{16}	5.6	
	- B3	190-210	4.7	50	1.0×10^{-9}	3.5		1.1×10^{13}	4.4	
	- B4	90-110	6.2	50	7.9×10^{-13}	4.2		1.6×10^{13}	4.3	
	- B5	190-210	5.6	50	2.0×10^{-10}	3.6		3.6×10^{12}	4.4	
	- C5	90-110	4.7	50	2.0×10^{-11}	3.8		3.0×10^{12}	3.9	
	AC (wearing course)									
	- B6	60-70	6.0	50	1.0×10^{-9}	3.6		3.9×10^{15}	4.9	
	- B7	60-70	6.5	50	5.0×10^{-20}	6.3		4.6×10^{18}	6.1	
	- B8	60-70	7.0	50	2.5×10^{-14}	4.9		1.0×10^{17}	5.2	
	DTM (base course) - B12	B54 CO Tar	5.2	32	5.0×10^{-10}	3.4		5.8×10^{11}	3.2	
	- B13	B54 CO Tar	5.2	50	1.0×10^{-7}	2.7		1.6×10^{10}	2.9	
	- B14	B54 CO Tar	5.2	68	3.1×10^{-9}	3.0		8.5×10^{9}	3.5	
	- C1	B54 CO Tar	5.5	50	1.3×10^{-10}	3.4		2.5×10^{11}	3.4	
	- C9	B50 CO Tar	5.2	50	5.0×10^{-7}	2.5		3.4×10^{9}	2.6	
	- C10	B50 CO Tar	5.2	68	1.3×10^{-9}	3.2		5.4×10^{9}	3.2	
	- D1	B54 VR Tar	5.2	32	1.6×10^{-15}	4.4		1.3×10^{16}	5.1	
	- D2	B54 VR Tar	5.2	68	5.0×10^{-8}	2.8		1.6×10^{10}	3.3	
	- D4	40-60	6.0	50	5.0×10^{-18}	5.8		7.5×10^{19}	6.4	
	MA (wearing course)									
	- B11	20	–	50	1.0×10^{-15}	5.5		1.5×10^{19}	5.5	
	- B11	15	–	68	5.0×10^{-13}	4.7		5.9×10^{16}	4.9	
	Gussasphalt (wearing course) - D9	40-60	8.0	50	4.0×10^{-19}	6.2		8.2×10^{21}	7.1	
Axial Load Controlled-Stress (Ref 60)*	HRA (base course) - E1	40-60	6.0	50	3.37×10^{-12}	4.1		9.49×10^{15}	5.4	
	- E2	40-60	6.0	77	4.26×10^{-9}	3.4		5.50×10^{10}	3.6	
	- E3	40-60	6.0	50	2.26×10^{-11}	3.9		4.25×10^{15}	5.1	
	- E4	40-60	6.0	50	1.94×10^{-7}	2.9		1.42×10^{17}	5.7	
	- E5	40-60	6.0	77	2.38×10^{-5}	2.5		7.98×10^{12}	4.3	
Indirect Tension Controlled-Stress (Ref 55)**	Project 2	AC-20	6.2-6.5	75				1.60×10^{12}	4.48	.92
	5	AC-10	6.9-7.2	75				1.88×10^{8}	2.66	.85
	8B	AC-20	4.6-5.3	75				2.29×10^{8}	2.32	.65
	8C	AC-20	5.6-5.9	75				3.93×10^{13}	4.44	.92
	17B(1)	AC-10 & 20	4.0	75				2.49×10^{6}	1.58	.73
	17B(2)	AC-10 & 20	4.0	75				2.63×10^{7}	2.18	.71
	25-97(1)	AC-20	6.0	75				4.52×10^{9}	3.18	.62
	25-97(2)	AC-20	6.0	75				3.10×10^{13}	4.61	.67
	25-97(3)	AC-20	3.5	75				2.55×10^{10}	3.39	.75
	25-97(S)	AC-20	5.0	75				8.18×10^{15}	5.08	.70
	25-100(1 & 2)	AC-20	4.5	75				4.20×10^{11}	4.21	.94
	25-100(3)	AC-20	3.2	75				1.36×10^{13}	4.68	.71
	25-100(S)	AC-20	4.7	75				9.80×10^{11}	4.14	.94

* K_2 converted from those associated with stress in N/mm^2 to those associated with stress in psi.

**K_2 is based on stress difference. In addition, all specimens in this group are from blackbase cores except those of Projects 25-97(S) and 25-100(S), which are from asphalt concrete cores.

APPENDIX B

SPECIMEN PROPERTIES AND STATIC TEST RESULTS

TABLE B-1. SPECIMEN PROPERTIES AND STATIC TEST RESULTS - 50°F

Aggregate	Asphalt Content, %	Bulk Density, pcf	Air Void Content, %	Tensile Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio
		139.2	9.8	187	242070	-.07
	4	139.2	9.8	232	260790	-.05
		138.7	10.1	189	226870	-.06
		140.9	7.4	336	349720	-.04
	5	142.4	6.4	401	328330	-.11
		141.6	6.9	358	411750	-.07
		145.7	2.8	621	601740	-.01
Limestone	6	145.5	3.0	612	598810	-.05
		144.6	3.6	518	585070	-.01
		146.4	1.0	588	453690	.04
	7	145.9	1.3	556	503600	-.01
		145.9	1.3	544	511420	.03
		144.6	.8	438	226430	-.03
	8	144.7	.8	494	351670	-0.00
		145.0	.6	415	297890	-.07
		136.5	10.3	137	124890	-.11
	4	136.5	10.3	171	177680	-.13
		136.1	10.6	175	200520	-.08
		139.3	7.1	305	306550	-.11
	5	139.9	6.8	299	294370	-.13
		140.4	6.4	332	343740	-.10
		141.7	4.2	167	436940	-.08
Gravel	6	141.2	4.6	434	441540	-.11
		144.0	2.7	519	589430	-.02
		144.6	.9	565	585480	.02
	7	144.3	1.1	176	757390	-.02
		144.8	.8	548	613300	-.01
		143.1	.6	446	592970	0.00
	8	143.2	.5	477	508840	.06
		143.4	.4	500	547250	-.03

TABLE B-2. SPECIMEN PROPERTIES AND STATIC TEST RESULTS - 75°F

Aggregate	Asphalt Content, %	Bulk Density, pcf	Air Void Content, %	Tensile Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio
		139.9	9.3	81	72982	.33
		138.5	10.3	75	66774	.20
		140.0	9.3	99	71168	.20
	4	139.4	9.6	77	77801	.21
		138.7	10.1	68	73877	.11
		139.9	9.3	89	63909	.09
		141.7	6.8	119	81174	.15
		142.9	6.1	130	94488	.08
		141.0	7.3	126	102040	.16
	5	143.3	5.8	129	117930	.05
		141.5	7.0	114	95501	.02
		141.7	6.8	129	104020	.04
		143.3	5.1	140	126690	.08
	5½	143.7	4.8	154	117950	.06
		142.6	5.5	141	101770	.09
		145.2	3.2	154	122990	.26
		145.3	3.1	156	119800	.44
		144.1	3.9	126	101060	.21
Limestone	6	145.0	3.3	119	112870	.22
		144.5	3.6	131	111760	.18
		144.2	3.8	133	119070	.19
		146.2	1.1	108	66122	.39
		146.5	.9	106	69884	.40
		145.8	1.4	108	88005	.50
		145.8	1.4	106	59839	.34
		145.7	1.5	111	100230	.34
		145.8	1.4	109	82091	.49
	7	146.1	1.2	107	68012	.32
		146.7	.8	117	93104	.53
		146.1	1.2	115	89417	.28
		146.3	1.1	125	99463	.44
		146.9	.6	104	88676	.24
		145.6	1.5	101	89946	.25
		146.2	1.1	104	83087	.22
		145.2	.4	57	64944	.43
		144.2	1.1	59	88570	.37
		144.5	.9	55	43568	.23
	8	144.9	.6	46	50044	.26
		144.8	.7	56	45724	.21
		144.4	.9	57	46057	.23

(Continued)

TABLE B-2. (Continued)

Aggregate	Asphalt Content, %	Bulk Density, pcf	Air Void Content, %	Tensile Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio
		137.3	9.8	73	83483	.24
		136.7	10.2	65	72357	.38
	4	135.7	10.8	62	70002	.33
		137.5	9.6	70	79636	.52
		137.1	9.9	58	62397	.34
		137.6	9.6	70	73460	.27
		139.1	7.3	107	117760	.19
		138.6	7.7	111	113940	.21
	5	138.5	7.7	110	116600	.18
		139.3	7.2	105	98655	.14
		138.4	7.8	95	88818	.29
		139.0	7.4	96	93000	.22
		142.2	3.9	152	177880	.26
		141.5	4.4	141	161320	.12
	6	141.7	4.2	139	165850	.13
		141.8	4.1	136	142070	.20
		143.7	2.9	145	150950	—
		141.5	4.4	141	137930	.13
		143.5	2.3	148	159660	.18
		144.4	1.8	147	—	—
Gravel	6½	144.1	2.0	147	205180	.23
		145.1	1.3	137	225700	.18
		144.0	2.0	144	200590	.22
		143.4	1.7	125	160200	.17
		143.0	2.0	127	166500	.29
		143.5	1.7	121	157240	.31
		143.5	1.7	141	170880	.25
		143.3	1.8	144	181590	.21
	7	143.5	1.7	143	165880	.37
		143.5	1.7	133	171360	.31
		143.7	1.5	131	156410	.22
		144.4	1.1	135	164380	.25
		143.2	1.9	149	191840	.24
		144.0	1.4	130	136480	.32
		143.8	1.5	137	144620	.28
		144.6	.9	119	122830	.15
		143.5	.4	108	139870	.29
		143.1	.7	110	130630	.24
	8	143.5	.4	97	115530	.25
		143.9	.1	94	95779	—
		143.9	.1	77	66330	.23
		143.3	.5	95	84503	.24

TABLE B-3. SPECIMEN PROPERTIES AND STATIC TEST RESULTS - 100°F

Aggregate	Asphalt Content, %	Bulk Density, pcf	Air Void Content, %	Tensile Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio
Limestone	4	138.4	10.3	27	26920	.73
		139.9	9.3	31	30063	.59
		139.3	9.7	24	21538	.56
	5	140.8	7.4	39	34030	.27
		141.8	6.8	40	33452	.29
		141.7	6.8	42	35489	.23
	6	146.9	2.0	48	31191	.86
		146.5	2.3	47	34107	.57
		145.7	2.8	47	35644	.73
	7	146.5	.9	19	15633	.67
		146.7	.8	20	16546	.49
		146.9	.6	20	16173	.70
	8	145.7	.1	6	4104	.46
		145.1	.5	6	4435	.64
		144.7	.8	9	6451	.59
Gravel	4	137.2	9.9	22	31107	.45
		136.3	10.4	22	26921	.45
		137.1	9.9	22	29229	.47
	5	139.2	7.3	31	30512	.34
		138.9	7.4	30	29812	.64
		140.1	6.6	31	31074	.35
	6	142.5	3.7	42	48051	.36
		141.6	4.3	35	50730	.50
		141.3	4.5	39	47951	.45
	7	143.9	1.4	29	36723	.29
		144.1	1.3	31	34277	.44
		143.6	1.6	3[illegible]	31841	.38
	8	143.3	.5	20	23867	.60
		143.3	.5	20	15828	.39
		143.7	.2	20	14435	.73

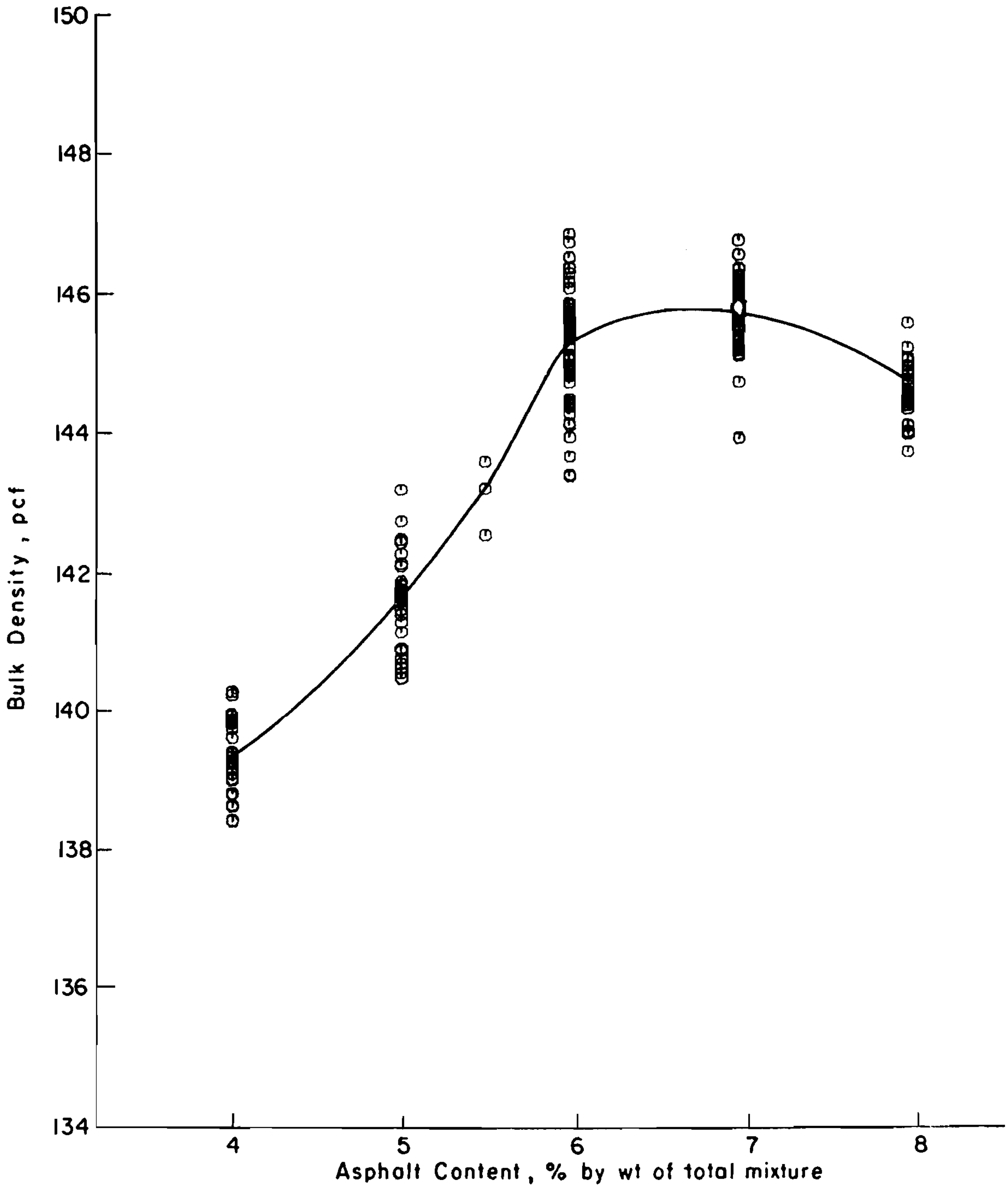

Fig B-1. Relationship between bulk density and asphalt content for limestone mixtures.

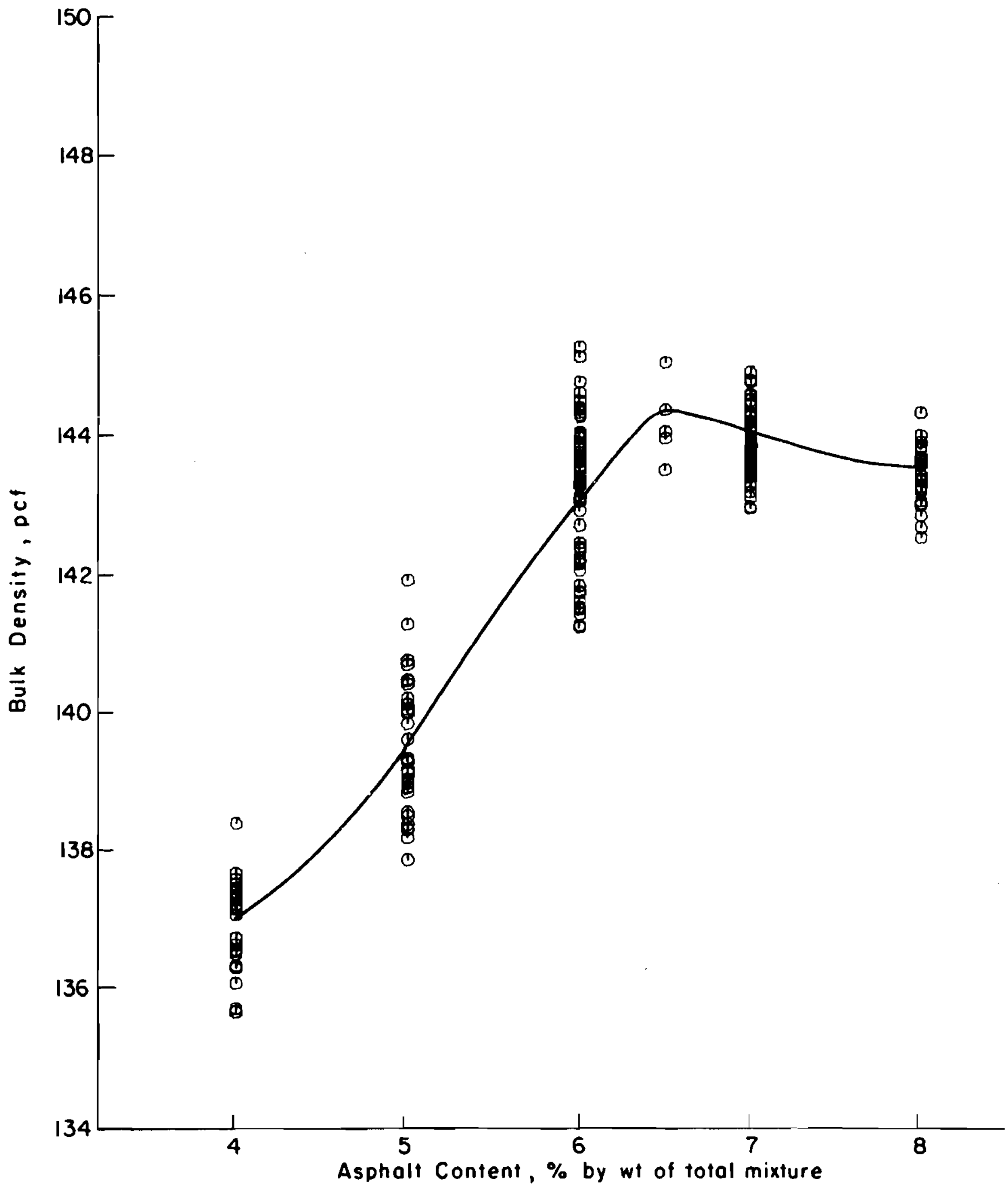

Fig B-2. Relationship between bulk density and asphalt content for gravel mixtures.

APPENDIX C

SPECIMEN PROPERTIES AND FATIGUE TEST RESULTS

TABLE C-1. SPECIMEN PROPERTIES AND FATIGUE TEST RESULTS

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				146.5	2.3	5856
			72	146.4	2.4	4381
				146.4	2.4	5977
				145.6	2.9	1889
50	Limestone	6	96	145.5	3.0	1552
				146.0	2.6	1559
				146.7	2.2	851
			120	144.5	3.6	737
				146.6	2.3	813
				143.4	3.1	3857
			72	144.1	2.6	5219
				143.6	2.9	3918
				143.1	3.3	916
50	Gravel	6	96	143.1	3.3	1006
				143.1	3.3	1182
				142.9	3.4	315
			120	141.8	4.1	274
				143.1	3.3	316
				139.5	9.6	520
			16	140.0	9.3	610
				140.3	9.1	745
75	Limestone	4				
				139.4	9.7	133
			24	139.1	9.9	145
				139.3	9.7	161
				139.9	9.3	211
				139.3	9.7	184

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
75	Limestone	4	32	139.4	9.7	90
				139.8	9.4	137
				138.9	10.0	99
			40	140.4	9.1	32
				139.7	9.5	99
				139.2	9.8	41
				139.1	9.9	39
75	Limestone	5	16	141.6	6.9	2308
				140.9	7.3	3461
				141.8	6.8	3716
				141.8	6.8	3925
				142.2	6.5	3258
			24	140.6	7.5	616
				142.2	6.5	904
				141.4	7.1	714
				142.0	6.7	1150
				140.6	7.6	1090
			32	140.7	7.5	398
				142.6	6.3	521
				141.7	6.8	493
				142.5	6.3	372
				141.5	7.0	505
			40	140.6	7.6	245
				141.2	7.1	193
				141.7	6.8	202
				141.9	6.7	289
				141.9	6.7	225
75	Limestone	6	16	145.8	2.8	7165
				145.9	2.7	7414
				144.3	3.8	7965
				145.0	3.3	10014
				145.6	2.9	6422
				143.5	4.3	9103

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				145.9	2.7	3670
				145.6	2.9	1948
				145.5	2.9	2172
			24	144.9	3.4	2383
				144.6	3.6	1898
				143.5	4.3	1420
				145.5	3.0	3058
				145.5	2.9	801
				143.8	4.1	805
				145.0	3.3	806
75	Limestone	6	32	145.8	2.8	978
				145.1	3.2	748
				145.2	3.2	915
				145.7	2.8	1125
				144.4	3.7	380
				144.6	3.5	343
				145.1	3.2	464
			40	145.0	3.3	379
				145.3	3.1	509
				146.0	2.6	465
				145.4	3.0	496
				144.1	2.6	190800
				145.3	1.7	178200
			8	145.3	1.7	200700
				145.9	1.3	322200
				145.9	1.3	206700
75	Limestone	7				
				146.0	1.2	6498
				145.2	1.8	6800
				145.6	1.5	11801
				145.5	1.6	11093
			16	145.7	1.5	8909
				146.0	1.2	8441
				146.2	1.1	6041
				145.8	1.4	7172

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				145.4	1.7	1402
				146.0	1.3	1812
			24	145.4	1.7	1412
				146.0	1.3	1942
				145.4	1.7	1645
				145.4	1.7	612
				145.8	1.4	488
			32	145.5	1.6	512
				146.0	1.3	851
				146.2	1.1	695
				146.0	1.3	509
75	Limestone	7				
				145.8	1.4	350
				146.4	1.0	490
				146.3	1.1	358
				145.5	1.6	475
			40	144.9	2.0	273
				145.9	1.3	287
				145.4	1.7	333
				145.9	1.3	315
			48	145.9	1.3	170
				146.0	1.2	206
			56	146.4	1.0	138
				146.4	1.0	120
				144.6	.9	4600
			16	144.6	.8	3891
				145.0	.6	3381
75	Limestone	8				
				144.7	.8	1250
				143.8	1.4	1066
			24	144.5	.9	1203
				144.2	1.1	858
				144.1	1.2	1313

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				144.6	.8	480
			32	144.1	1.2	360
				145.2	.4	323
75	Limestone	8				
				145.4	.3	190
			40	144.1	1.2	180
				145.0	.6	212
				137.4	9.7	332
			16	137.3	9.8	346
				137.3	9.8	321
				137.2	9.8	129
				137.4	9.7	135
75	Gravel	4	24	138.4	9.1	98
				136.6	10.3	77
				136.3	10.4	66
				136.3	10.4	32
			32	135.7	10.9	32
				137.7	9.5	56
				136.6	10.2	24
			40	137.2	9.8	15
				137.1	9.9	24
				140.5	6.4	1461
				140.5	6.4	1640
			16	140.0	6.7	1290
				138.2	7.9	1594
				139.0	7.3	1558
75	Gravel	5				
				141.3	5.8	586
				140.0	6.7	456
			24	140.2	6.6	492
				139.3	7.2	402
				138.3	7.8	278

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Aspahlt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
75	Gravel	5	32	140.1	6.7	198
				140.7	6.2	265
				139.0	7.3	114
				138.3	7.8	156
				139.3	7.2	134
				141.9	5.4	242
			40	138.9	7.4	112
				139.6	7.0	105
				140.8	6.2	121
				138.8	7.5	100
				137.9	8.1	78
75	Gravel	6	16	144.0	2.7	7521
				142.2	3.9	10292
				142.4	3.8	5234
				143.6	3.0	4670
				143.5	3.0	7785
				143.9	2.8	9709
				143.7	2.9	11231
			24	144.3	2.5	1217
				144.0	2.7	1874
				142.7	3.5	2249
				141.5	4.4	1123
				144.4	2.4	1724
				143.1	3.3	1453
				143.3	3.1	1594
			32	143.4	3.1	749
				144.4	2.5	793
				144.6	2.3	932
				144.5	2.3	720
				143.8	2.8	1160
				141.4	4.4	440
				144.8	2.2	866

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				143.8	2.8	416
				143.3	3.2	271
				143.3	3.1	315
75	Gravel	6	40	142.1	4.0	270
				143.7	2.9	3[illegible]1
				142.2	3.9	213
				142.2	3.9	300
				143.6	1.6	199560
				143.6	1.6	270780
			8	143.8	1.5	329100
				143.5	1.7	573720
				143.5	1.7	378000
				144.5	1.0	3648
				143.0	2.0	3307
				143.7	1.6	5245
				143.9	1.4	6137
75	Gravel	7	16	143.1	1.9	6987
				144.1	1.3	6496
				143.4	1.7	10807
				144.8	.8	6293
				143.8	1.5	7865
				143.9	1.4	784
				144.0	1.4	784
				144.1	1.3	850
				144.9	.7	1317
			24	144.5	1.0	1719
				143.7	1.5	1286
				144.0	1.4	1537
				144.0	1.4	2500
				144.0	1.4	3012

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
				143.9	1.4	273
				144.2	1.2	586
				143.3	1.8	548
				144.6	.9	536
			32	143.8	1.5	609
				143.8	1.5	1095
				143.7	1.5	866
				143.8	1.5	296
				143.5	1.7	258
75	Gravel	7				
				144.1	1.2	313
				143.7	1.5	292
				143.8	1.5	266
				143.8	1.5	525
			40	144.2	1.2	690
				144.2	1.2	650
				144.2	1.2	500
				144.3	1.1	163
				143.7	.2	4063
			16	143.7	.2	1524
				142.7	.9	7653
				143.6	.3	587
				143.4	.4	551
			24	144.0	.0	675
				143.9	.0	913
				142.6	1.0	1647
75	Gravel	8				
				143.7	.2	441
			32	142.9	.8	513
				143.0	.7	180
				143.7	.2	220
			40	143.6	.3	221
				144.4	.0	186

(Continued)

TABLE C-1. (Continued)

Testing Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Bulk Density, pcf	Air Void Content, %	Fatigue Life, Cycles
100	Limestone	6	8	145.8	2.8	2417
				147.1	1.9	2350
				145.6	2.9	1685
			16	145.8	2.8	294
				145.9	2.7	363
				146.7	2.2	326
			24	145.1	3.2	131
				146.0	2.6	113
				146.2	2.5	104
100	Gravel	6	8	145.1	1.9	1332
				145.3	1.8	1486
				143.7	2.9	765
			16	142.4	3.8	152
				143.1	3.3	167
				144.4	2.4	213
			24	143.4	3.1	54
				144.3	2.5	66
				144.4	2.4	55

APPENDIX D

THEORY, WORKING EQUATIONS, AND DATA REDUCTION TECHNIQUES

APPENDIX D. THEORY, WORKING EQUATIONS, AND DATA REDUCTION TECHNIQUES

THEORY

The indirect tensile test involves the loading of a cylindrical specimen with compressive loads distributed along two opposite generators as shown in Fig D-1. This type of loading condition creates a relatively uniform tensile stress distribution perpendicular to and along the diametral plane which contains the applied load. Failure usually occurs by splitting along the loaded plane (Fig D-1).

The theory of stress distribution was originally developed by Hertz for line loads (Ref 20) and later modified by Hondros to account for distribution of loads over strips of finite width (Ref 23), as illustrated in Fig D-2. The equations, which are valid for conditions of both plane stress (discs) and plane strain (cylinders), are as follows:*

(a) Stresses along the vertical diameter

tangential stress:

$$\sigma \epsilon y = \frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1-2r^2/R^2\cos 2\alpha + r^4/R^4)} - \tan^{-1}\left(\frac{(1+r^2/R^2)}{(1-r^2/R^2)}\right)\right] \qquad \text{(D-1)}$$

radial stress:

$$\sigma ry = \frac{-2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1+2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\left(\frac{(1+r^2/R^2)}{(1-r^2/R^2)}\tan\alpha\right)\right] \qquad \text{(D-2)}$$

shear stress:

$$\tau r\theta = 0 \qquad \text{(D-3)}$$

(b) Stresses along the horizontal diameter

tangential stress:

$$\sigma\theta x = \frac{-2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1+2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\left(\frac{(1-r^2/R^2)}{(1+r^2/R^2)}\tan\alpha\right)\right] \qquad \text{(D-4)}$$

radial stress:

$$\sigma\theta x = \frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1+2r^2/R^2\cos 2\alpha + r^4/R^4)} - \tan^{-1}\left(\frac{(1-r^2/R^2)}{(1+r^2/R^2)}\tan\alpha\right)\right] \qquad \text{(D-5)}$$

shear stress:

$$\tau r\theta = 0 \qquad \text{(D-6)}$$

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*Definitions of symbols used in the equations are contained in Fig D-2.

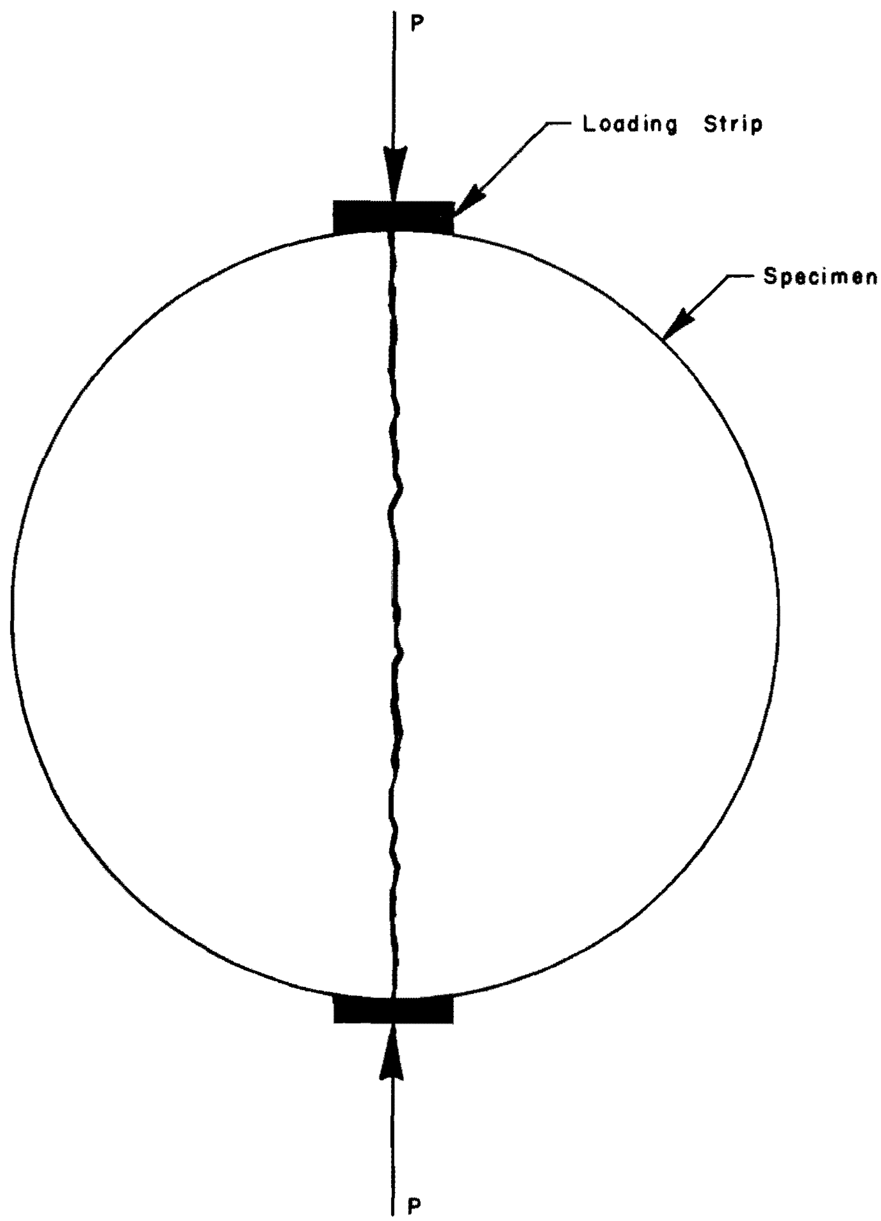


Fig D-1. The indirect tensile test.

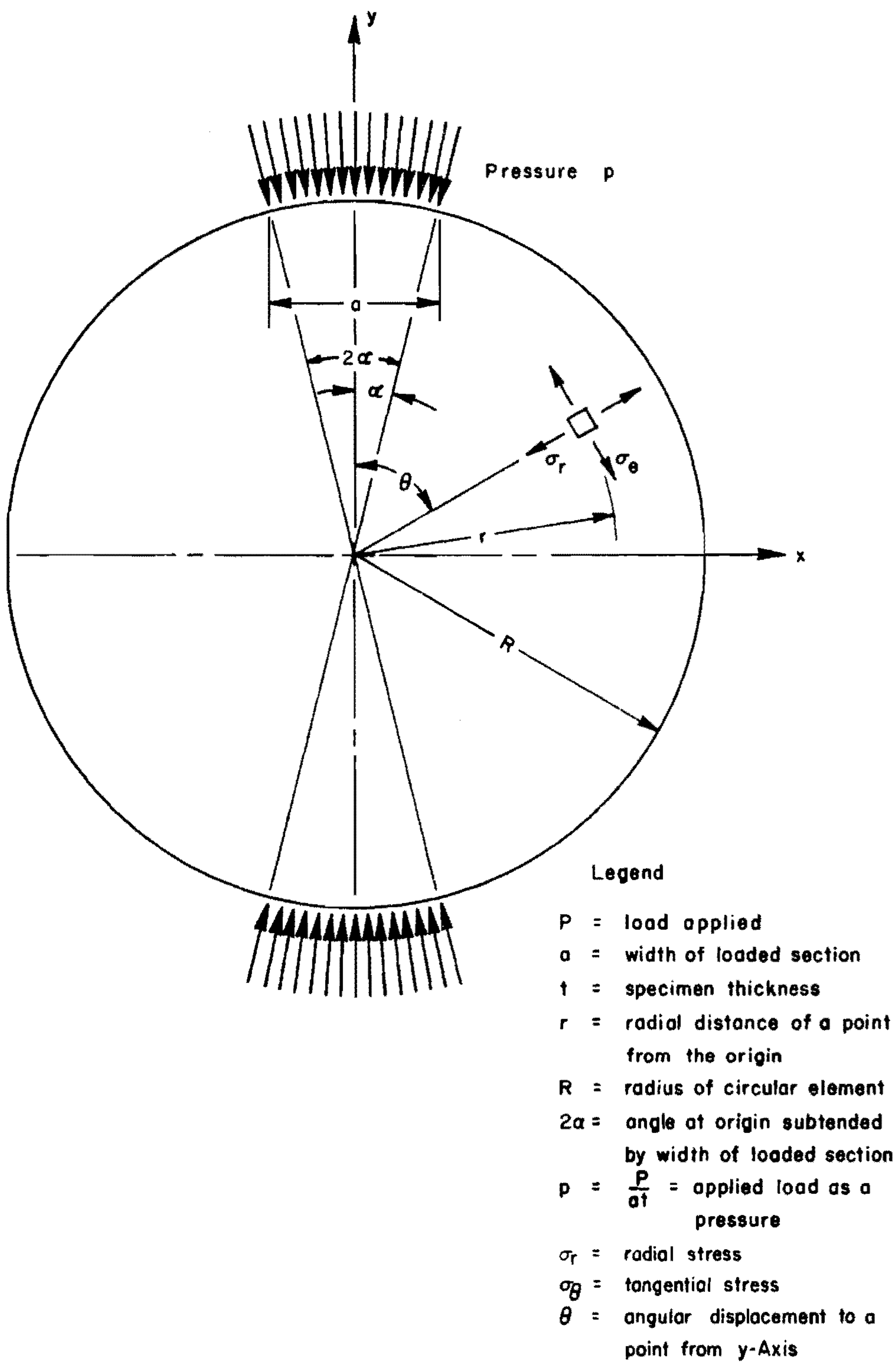


Fig D-2. Notation for polar stress components in a circular element compressed by short strip loadings (Ref 23).

The stresses along the principal planes corresponding to the horizontal and vertical diameters for a loading strip width less than D/10 (D = diameter), are plotted in Fig D-3. For this case the equations for the stresses at the center would reduce to

$$\sigma\theta y = \sigma rx = \frac{2P\alpha}{\pi at} = \frac{2P}{\pi tD} \quad \text{(tensile)} \qquad \text{(D-7)}$$

and

$$\sigma\theta x = \sigma ry = -\frac{6P\alpha}{\pi at} = -\frac{6P}{\pi tD} \quad \text{(compressive)} \qquad \text{(D-8)}$$

where

$r/R = 0$ and

$\alpha = a/D$, approximately.

WORKING EQUATIONS

Properties evaluated in this study include the following:

(a) modulus of elasticity,

(b) Poisson's ratio,

(c) tensile strength,

(d) tensile strain, and

(e) permanent strain.

The above properties were estimated by assuming the validity of the basic elastic equations, Hondros' equations presented earlier (Eq D-1 through D-6), and the condition of plane stress. The final forms of the equations utilized were as follows:

<u>Modulus of Elasticity E</u>

$$E = \frac{P}{X_T}\left[\int_R^R \frac{\sigma rx}{P} - \nu\int_R^R \frac{\sigma\theta x}{P}\right] \qquad \text{(D-9)}$$

where

$\frac{P}{X_T}$ = the least squares line of best fit between load P and total deformation X_T for loads in the initial linear (or nearly linear) portion of the load-deformation curve. In the case of fatigue test, this value is simply the ratio of the repeated load to the horizontal deformation at the particular

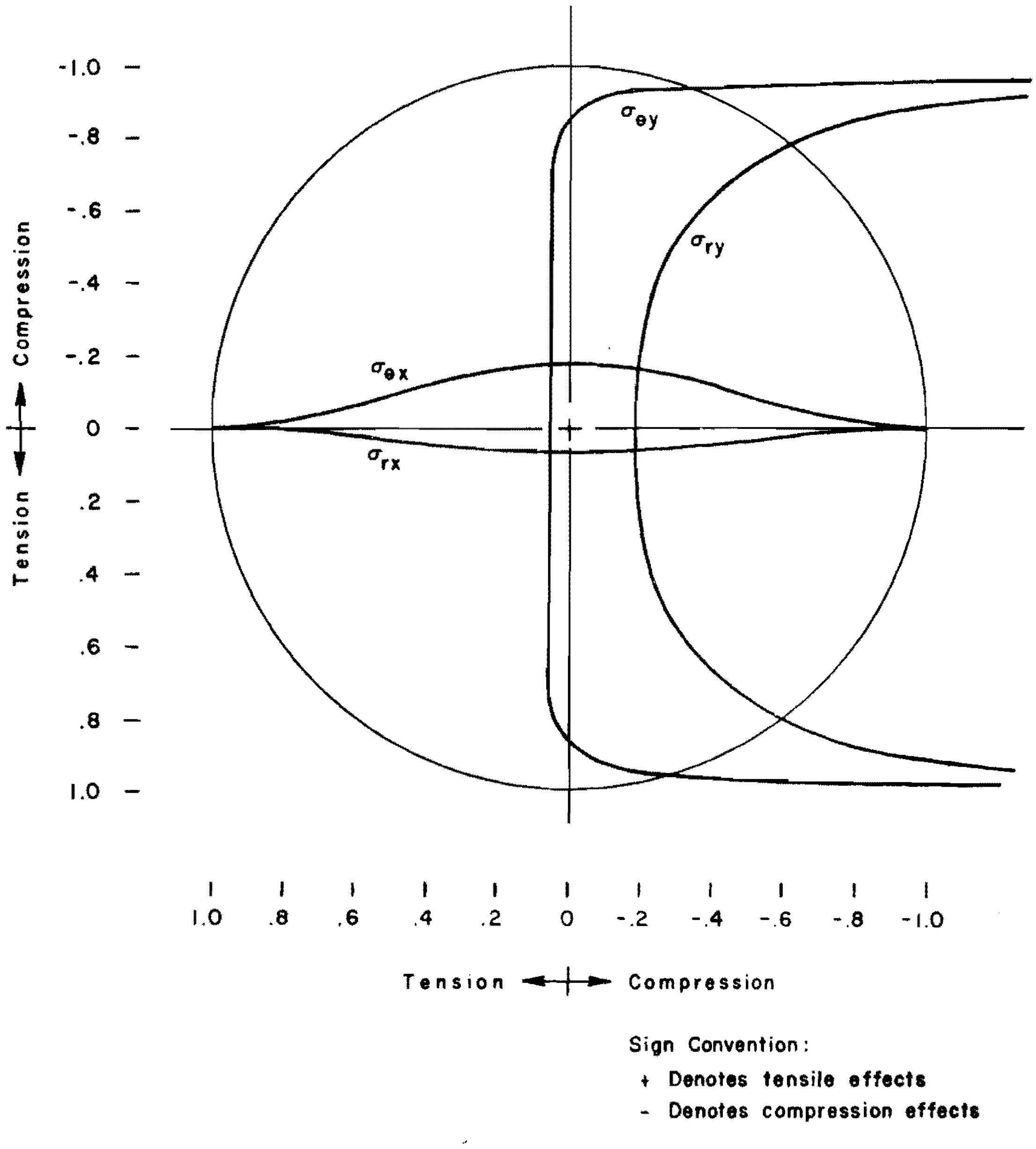


Fig D-3. Stress distribution along the principal axes for loading strip width less than D/10; $(2P/\pi at = 1)$.

cycle number where the value of E is being determined. X_T may then be H_{RI}, H_{RT}, H_T, or H_{TT} as defined in Chapter 3, depending on which type of modulus was being evaluated.

ν = Poisson's ratio, as determined from the equation expressed below.

$\int_{R}^{R} \frac{\sigma rx}{P}$ and $\int_{R}^{R} \frac{\sigma\theta x}{P}$ = the integration of the unit stresses σrx and $\sigma\theta x$ (See equations D-4 and D-5).

R = radius of specimen.

r = radial distance of an element from the center (Fig D-2).

Poisson's Ratio ν

$$\nu = \frac{\left[\int_{-R}^{R} \sigma ry + \bar{R} \int_{-R}^{R} \sigma rx\right]}{\left[\bar{R} \int_{-R}^{R} \sigma\theta x + \int_{-R}^{R} \sigma\theta y\right]} \tag{D-10}$$

where

$\int_{-R}^{R} \sigma ry$ and $\int_{-R}^{R} \sigma rx$ = integration of radial stresses in the y and x-directions, respectively (Eqs D-2 and D-5),

$\int_{-R}^{R} \sigma\theta x$ and $\int_{-R}^{R} \sigma\theta y$ = integration of radial stresses in the x and y-directions, respectively (Eqs D-4 and D-1), and

$\bar{R} = \frac{Y_T}{X_T}$ = the least squares line of best fit between vertical deformation Y_T and the corresponding horizontal deformation X_T for loads in the linear or nearly linear portion of the load-deformation curves (generally up to about 50 percent of maximum load). In the case of fatigue test, $\bar{R}$ = ratio of vertical deformation to the corresponding horizontal deformation at the desired cycle number. X_T and Y_T may be any of the deformations defined in Chapter 3 for various forms of ν.

Tensile Strength S_T

From Eq D-1 or D-5, the tensile stress perpendicular to the applied load at the center of the specimen $(r = 0)$ is

$$S_T = \frac{2P_{max}}{\pi a t} (\sin 2\alpha - \alpha) \tag{D-11}$$

where

P_{max} = the load at the first inflection point of the load-horizontal deformation curve. In the case of fatigue tests, P_{max} is the repeated load and S_T becomes the applied repeated stress and

a, t, and α are as defined in Fig D-2.

Tensile Strain ϵ

$$\epsilon = \frac{X}{\ell} \frac{\left[\int_{-\ell/2}^{\ell/2} \frac{\sigma rx}{P} - \nu \int_{-\ell/2}^{\ell/2} \frac{\sigma\theta x}{P}\right]}{\left[\int_{-R}^{R} \frac{\sigma rx}{P} - \nu \int_{-R}^{R} \frac{\sigma\theta x}{P}\right]} \tag{D-12}$$

where

X = total horizontal deformation and in the case of fatigue tests is the deformation at the required cycle number; this may be H_{RI}, H_{RT}, H_T, or H_{TT} as defined in Chapter 3.

ℓ = length over which strain is estimated (ℓ = 0.005 inch for this study)*.

ν = Poisson's ratio.

$\frac{\sigma rx}{P}$ and $\frac{\sigma\theta x}{P}$ = unit stresses (Eqs D-5 and D-4).

Permanent Strain ϵ_p

The total permanent strain was calculated using the simple relationship

$$\epsilon_p = \epsilon_{TT} - \epsilon_{RT} \tag{D-13}$$

*It was found that center strain was best evaluated for ℓ less than 0.1 inch (Ref 17).

where

ϵ_{TT} = tensile strain using equation D-12 with X = cumulative total deformation (defined in Chapter 3).

ϵ_{RT} = tensile strain using equation D-12 with X = total recovered (resilient) deformation (defined in Chapter 3).

For the derivation and verification of Eq D-9, D-10, D-11, and D-12, reference should be made to CFHR Research Report 98-7 (Ref 17).

DATA REDUCTION TECHNIQUES - COMPUTER PROGRAMS

The utilization of test results in the evaluation of the various tensile and elastic properties could be very involved without the development and availability of certain computer programs. The functional aspects of these programs are described briefly below.

MODLAS 9 - Asphalt

This program, originally formulated by William Hadley and modified by Joe Kozuh and Bryant Marshall (Ref 41) was utilized unabridged for the static tests. It performed all the integrations and regressions appearing in Eqs D-9 through D-12. The output from the program includes the computed values of tensile strength, tensile strains, modulus of elasticity, and Poisson's ratio.

MODLAS 10 - Fatigue

This program, developed for this study, was merely a modification of Modlas 9 for repeated-load tests. It calculates resilient and permanent strains, modulus of elasticity, and Poisson's ratio at any desired cycle number.

Step-01

This statistical computer program for stepwise multiple regression was originally BMD-2R modified by Mrs. L. Stewart and Messrs. W. A. Wilson, R. S. Walter, and J. A. Kozuh for the CFHR. With appropriate transgeneration cards, the program was utilized for curvilinear and other desirable regressions. The output from the program includes multiple R, standard error for residuals, analysis of variance table, regression coefficient and standard error for variables in the equation, and plots of residuals versus input variables.

Harvey ANOVA

This program, written under the direction of Dr. W. R. Harvey at Ohio State University, calculates an analysis of variance for a factorial arrangement of treatments. The effects of all classification variables, first order interactions, and continuous independent variables are included in the analysis of variance. A good aspect of the program is that the number of observations per cell may be unequal and entire cells may be missing, with certain rearrangement of cells.

Other computer programs utilized include plotting programs specifically written for this study.

APPENDIX E

RELATIONSHIPS BETWEEN FATIGUE LIFE AND AIR VOID CONTENT

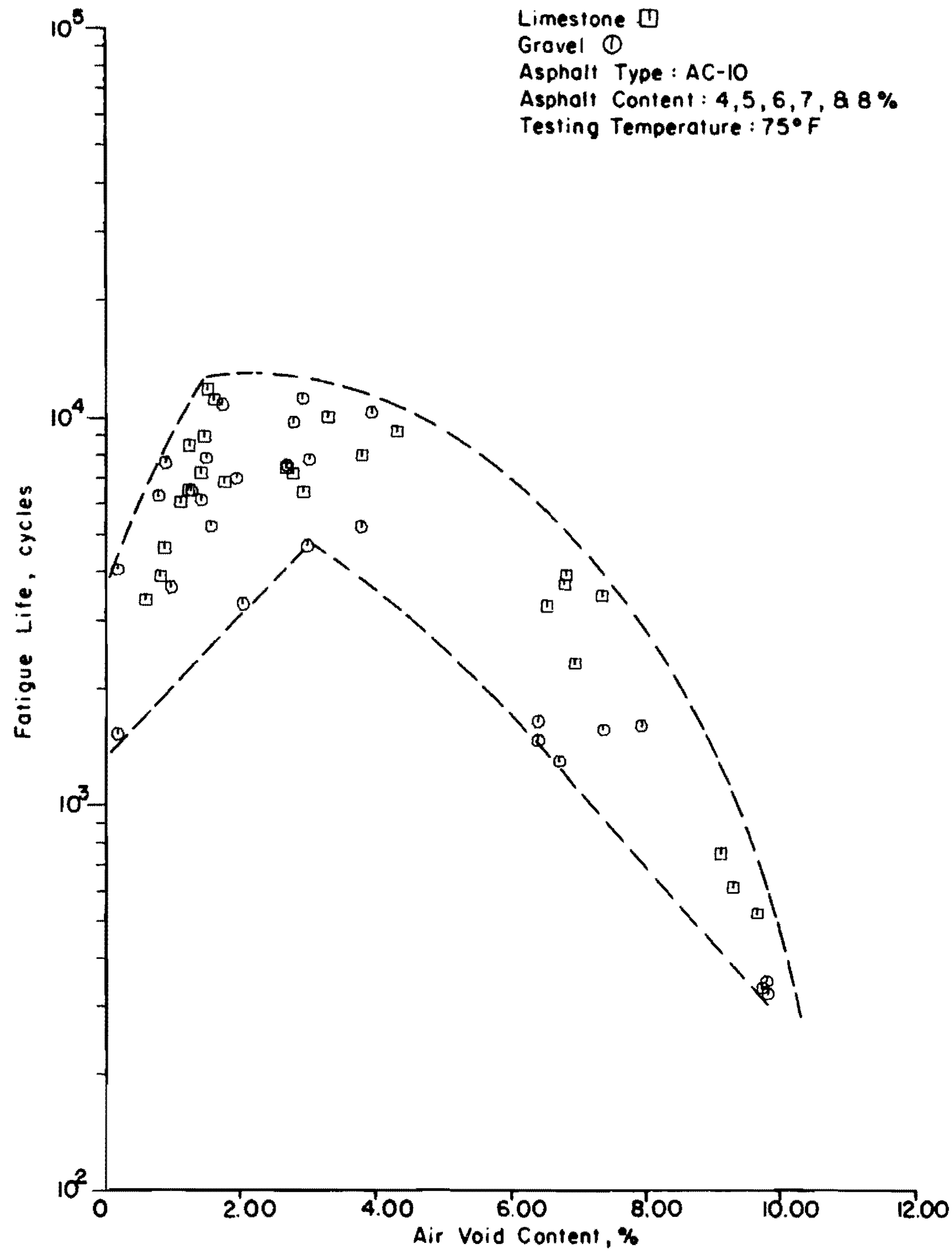


Fig E-1. Relationship between fatigue life and air void content - 16 psi.

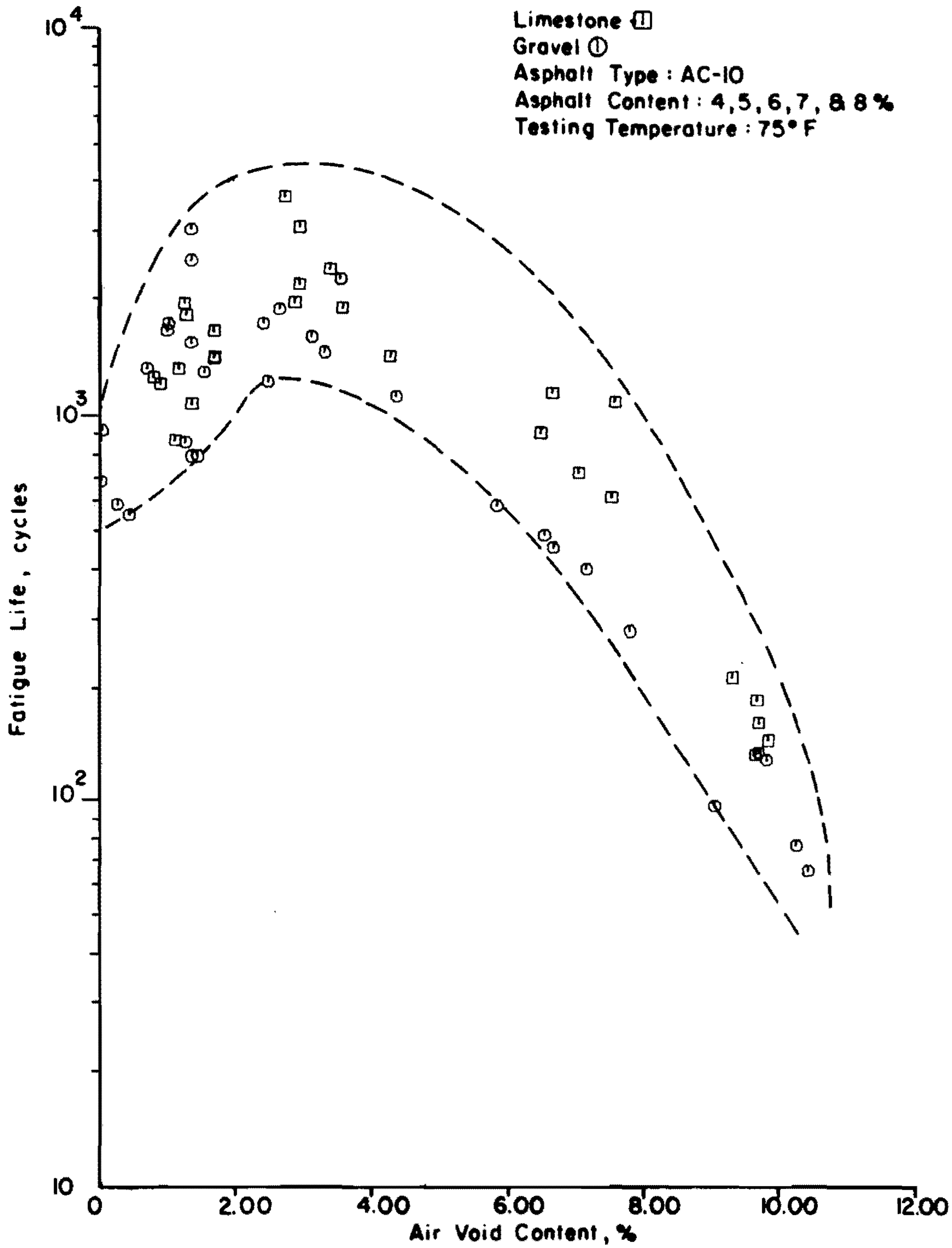


Fig E-2. Relationship between fatigue life and air void content - 24 psi.

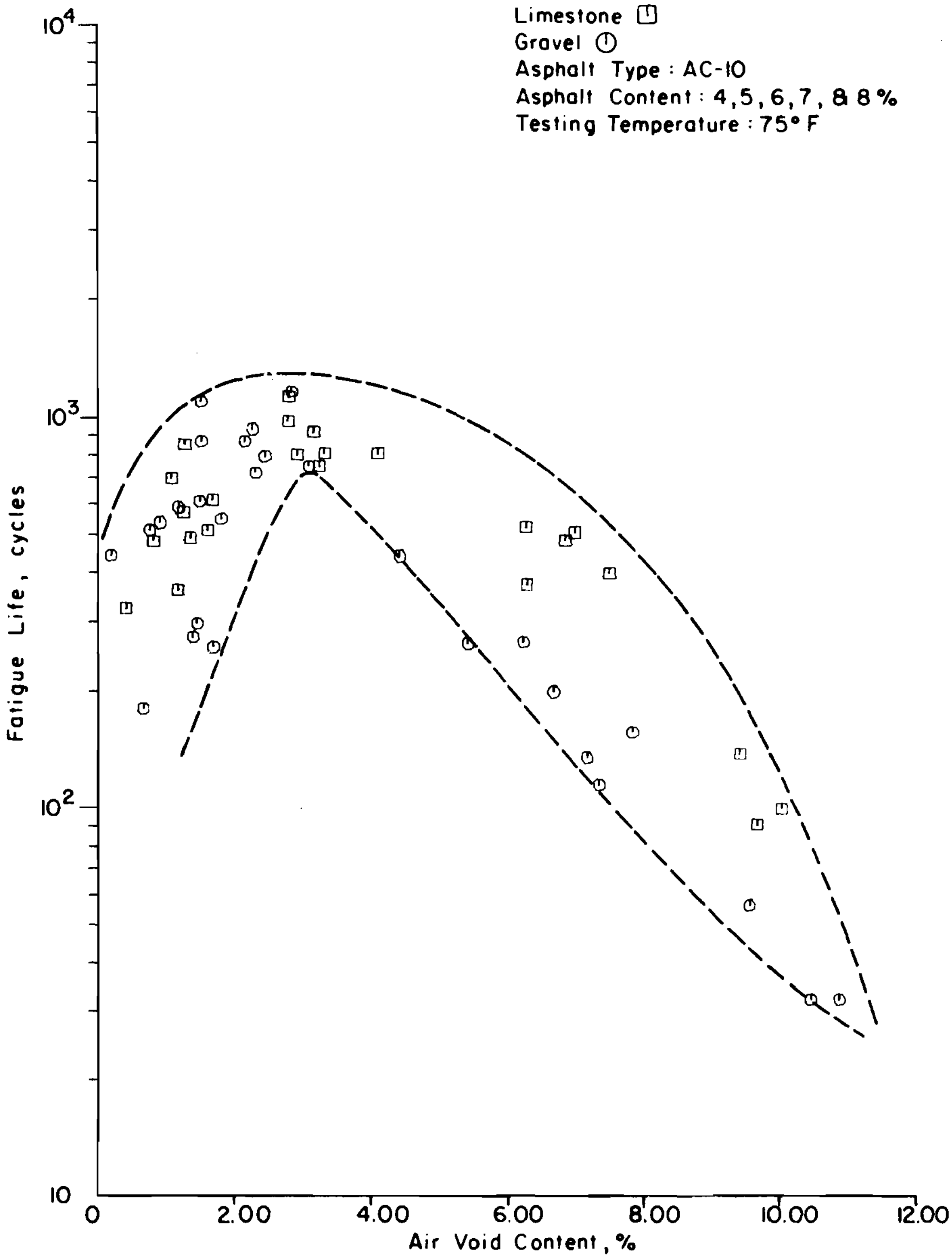


Fig E-3. Relationship between fatigue life and air void content - 32 psi.

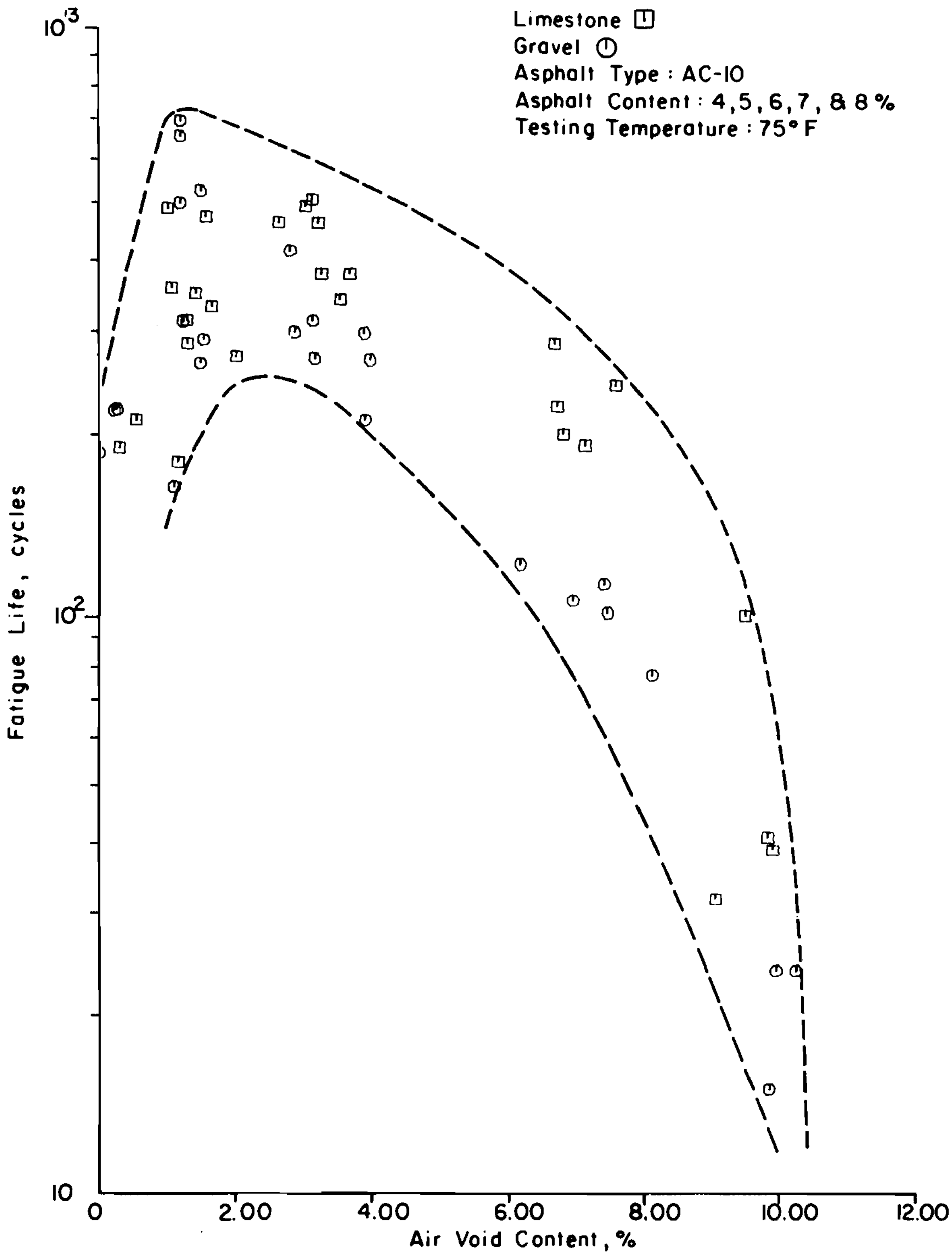


Fig E-4. Relationship between fatigue life and air void content - 40 psi.

APPENDIX F

LEAST SQUARES REGRESSION COEFFICIENTS FOR THE LINEAR PORTIONS OF THE RELATIONSHIPS BETWEEN RESILIENT MODULI AND NUMBER OF LOAD REPETITIONS

TABLE F-1. LEAST SQUARES REGRESSION COEFFICIENTS FOR THE LINEAR PORTIONS OF THE RELATIONSHIPS BETWEEN RESILIENT MODULI AND NUMBER OF LOAD REPETITIONS.

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Points	Instantaneous Resilient Modulus Intercept, psi	Instantaneous Resilient Modulus Slope, psi/cycle	Instantaneous Resilient Modulus R	Total Resilient Modulus Intercept, psi	Total Resilient Modulus Slope, psi/cycle	Total Resilient Modulus R
				8	714,810	10	0.41	576,326	12	0.64
			72	13	828,366	30	0.60	689,493	25	0.82
				14	–	–	–	652,686	6	0.54
				–	–	–	–	–	–	–
	Limestone	6	96	11	637,745	6	0.15	540,047	21	0.86
				10	676,148	7	0.11	577,291	20	0.74
				11	857,516	209	0.85	668,698	142	0.88
			120	8	764,651	41	0.31	584,178	31	0.23
				8	1,038,365	507	0.88	777,722	313	0.92
50				8	1,064,074	108	0.99	904,951	77	0.98
			72	8	–	–	–	779,415	22	0.72
				8	1,185,912	50	0.92	1,024,586	56	0.91
				14	1,051,202	674	0.97	827,034	494	0.99
	Gravel	6	96	7	1,143,533	296	0.76	994,252	263	0.93
				8	1,048,468	150	0.90	881,765	117	0.92
				13	893,057	523	0.80	717,713	487	0.97
			120	13	1,050,733	982	0.92	783,303	662	0.96
				8	753,263	85	0.52	694,493	311	0.93
				8	269,419	675	0.97	190,648	409	0.98
		4	24	10	291,954	199	0.59	205,294	214	0.76
				9	317,882	519	0.88	216,540	271	0.94
				10	391,948	52	0.94	257,127	30	0.97
			16	13	366,076	10	0.58	253,314	14	0.93
				13	374,623	5	0.44	258,783	10	0.90
				11	334,747	209	0.97	214,628	102	0.99
			24	23	353,189	121	0.94	233,677	67	0.95
				23	349,016	63	0.72	226,460	58	0.94
		5		15	348,932	112	0.69	241,061	86	0.82
			32	15	555,565	444	0.85	350,151	255	0.90
				15	368,098	196	0.91	259,326	144	0.98
				14	407,955	450	0.78	294,675	326	0.87
			40	11	451,829	824	0.91	303,805	458	0.96
				13	379,049	347	0.74	273,663	292	0.88
				13	604,351	25	0.89	336,043	12	0.97
				16	237,531	2	0.55	173,189	2	0.57
			16	11	233,959	4	0.80	163,536	3	0.90
				11	221,200	3	0.74	162,662	3	0.90
				10	237,580	3	0.69	172,745	3	0.57
				9	515,779	12	0.28	341,417	11	0.74
				–	–	–	–	–	–	–
			24	–	–	–	–	–	–	–
				11	277,697	22	0.91	198,298	15	0.91
				12	274,585	17	0.77	196,204	15	0.89
75	Limestone	6		15	282,235	29	0.43	198,967	26	0.83
				14	287,910	39	0.81	201,035	26	0.85
			32	15	264,728	18	0.50	194,222	36	0.82
				–	–	–	–	–	–	–
				14	269,607	26	0.68	194,172	34	0.91
				14	333,847	90	0.83	229,667	70	0.90
				9	347,108	309	0.95	234,665	176	0.94
			40	11	372,571	97	0.81	259,176	129	0.97
				13	339,930	126	0.79	231,842	129	0.94
				13	396,052	254	0.97	246,665	140	0.98

(Continued)

TABLE F-1. (Continued)

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Points	Instantaneous Resilient Modulus Intercept, psi	Instantaneous Resilient Modulus Slope, psi/cycle	Instantaneous Resilient Modulus R	Total Resilient Modulus Intercept, psi	Total Resilient Modulus Slope, psi/cycle	Total Resilient Modulus R
				10	874,360	77	0.92	365,964	20	0.95
				11	548,744	33	0.98	325,307	15	0.97
			16	16	629,793	19	0.96	356,437	4	0.42
				—	—	—	—	—	—	—
				15	258,492	3	0.87	172,922	3	0.21
				11	240,489	3	0.77	168,581	3	0.92
				10	548,728	106	0.91	338,791	76	0.96
		7	24	5	571,287	93	0.95	351,307	62	0.98
				11	610,858	154	0.97	372,038	104	0.99
				9	680,007	244	0.94	438,147	306	0.92
			32	16	635,284	350	0.65	386,394	324	0.95
				10	723,701	625	0.96	385,756	208	0.92
				12	511,354	216	0.91	344,776	151	0.97
				11	524,810	322	0.77	351,342	320	0.92
			40	15	498,806	302	0.83	345,386	301	0.95
				15	623,245	671	0.94	348,673	220	0.99
			16	11	414,059	13	0.46	259,693	11	0.78
				13	488,574	48	0.97	280,799	20	0.96
				17	427,195	60	0.78	265,688	31	0.80
75	Limestone	8		18	464,780	90	0.79	250,855	24	0.86
			24	13	500,962	129	0.88	316,840	93	0.98
				14	313,957	109	0.86	202,631	63	0.89
				14	332,806	69	0.89	218,980	45	0.89
				8	425,950	670	0.80	335,947	1157	0.96
		4	24	8	298,539	396	0.70	209,016	377	0.95
				7	329,360	827	0.62	206,542	273	0.63
				12	492,429	117	0.93	312,531	64	0.91
			16	13	502,402	110	0.92	303,822	47	0.92
				10	492,786	106	0.84	290,469	56	0.93
				7	582,250	60	0.55	401,100	160	0.85
			24	18	454,171	452	0.82	300,342	281	0.93
		5		33	341,610	158	0.81	245,676	101	0.79
				13	494,338	893	0.94	317,966	520	0.97
			32	15	521,977	740	0.43	341,620	382	0.91
				9	456,265	942	0.38	292,719	585	0.97
				8	547,995	1814	0.95	322,029	844	0.93
			40	8	464,273	1212	0.71	307,928	692	0.90
				9	542,169	1403	0.92	348,201	1064	0.98
				13	635,943	47	0.90	400,132	29	0.99
			16	12	240,257	2	0.49	192,414	4	0.94
				14	236,983	1	0.66	191,033	3	0.94
				15	239,979	1	0.10	189,750	1	0.55
				—	—	—	—	—	—	—
				13	425,038	46	0.60	320,404	56	0.80
			24	10	446,908	74	0.92	319,421	62	0.99
				16	284,048	18	0.74	216,418	26	0.96
				14	286,432	19	0.76	212,193	20	0.92
75	Gravel	6		11	162,009	51	0.80	104,660	37	0.95
				15	331,385	91	0.96	250,192	84	0.95
			32	15	379,726	94	0.84	277,357	85	0.97
				14	340,075	77	0.80	249,308	54	0.88
				15	330,533	73	0.94	246,027	58	0.93

(Continued)

TABLE F-1. (Continued)

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Number of Points	Instantaneous Resilient Modulus			Total Resilient Modulus		
					Intercept, psi	Slope, psi/cycle	R	Intercept, psi	Slope, psi/cycle	R
				15	380,998	144	0.79	275,263	107	0.89
				14	386,795	367	0.86	264,923	232	0.95
		6	40	8	376,947	214	0.86	269,471	201	0.91
				12	345,859	147	0.71	243,889	143	0.89
				13	381,880	236	0.77	277,091	229	0.97
				9	623,498	66	0.88	371,761	37	0.91
				10	602,372	56	0.88	359,895	39	0.96
			16	13	608,297	36	0.69	344,615	18	0.93
				8	645,773	37	0.96	426,226	21	0.81
				15	268,641	3	0.84	201,342	4	0.89
				10	267,143	2	0.79	194,199	3	0.96
				9	589,641	302	0.95	367,759	197	0.99
				8	736.077	522	0.99	418,868	225	0.97
		7		6	633,664	316	0.97	394,904	203	0.94
			24	12	312,025	47	0.94	222,914	36	0.98
				13	328,821	35	0.96	232,204	29	0.96
				11	304,898	17	0.62	223,491	26	0.89
75	Gravel			15	329,287	41	0.88	242,373	47	0.98
				18	677,399	926	0.77	377,871	596	0.99
				15	339,425	142	0.95	235,822	69	0.90
			32	14	341,339	111	0.82	250,104	106	0.97
				15	325,910	111	0.80	230,711	93	0.96
				15	359,595	137	0.90	263,430	120	0.99
				12	419,476	239	0.83	296,539	218	0.93
			40	16	423,172	373	0.90	299,269	368	0.98
				11	446,446	430	0.95	297,378	306	0.95
			16	8	576,849	196	0.96	327,751	85	0.96
		8		12	490,067	272	0.87	313,246	208	0.98
			24	19	466,024	473	0.95	296,987	283	0.98
				23	412,591	179	0.83	260,804	108	0.95
				8	161,320	32	0.98	104,549	19	0.96
			8	7	136,952	7	0.68	95,687	5	0.49
				5	126,359	14	0.75	86,665	4	0.48
				10	186,961	148	0.80	131,724	99	0.95
	Limestone	6	16	10	172,069	101	0.82	114,353	53	0.94
				13	168,710	155	0.96	115,501	92	0.90
				9	197,665	264	0.93	143,434	253	0.98
			24	8	279,550	905	0.95	183,456	427	0.95
				7	—	--	—	163,629	456	0.97
100				8	216,910	58	0.97	161,994	44	0.93
			8	8	218,324	65	0.98	165,029	54	0.99
				8	204,633	121	0.98	142,919	75	0.99
				11	249,723	477	0.90	178,319	312	0.95
	Gravel	6	16	12	267,819	743	0.91	171,385	331	0.99
				8	284,891	660	0.97	183,230	307	0.98
				3	375,142	4899	0.96	196,639	986	0.99
			24	4	289,400	2014	0.95	191,812	640	0.98
				4	272,173	2101	0.99	201,783	1347	1.00

APPENDIX G

VALUES OF REPEATED-LOAD RESILIENT MODULUS AND MODULUS OF CUMULATIVE TOTAL DEFORMATION

TABLE G-1. VALUES OF REPEATED-LOAD RESILIENT MODULUS AND MODULUS OF CUMULATIVE TOTAL DEFORMATION

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Instantaneous Resilient Modulus E_{RI}, psi	Total Resilient Modulus E_{RT}, psi	Modulus of Cumulative Total Deformation E_{TT}, psi	$\frac{E_{RT}}{E_{RI}}$, %	$\frac{E_{TT}}{E_{RI}}$, %
50	Limestone	6	72	697,665	551,075	20,369	79	2.9
				756,300	637,616	23,170	84	3.1
				754,879	639,649	26,937	85	3.6
			96	631,804	513,753	50,491	81	8.0
				630,565	523,514	39,705	83	6.3
				662,554	559,507	35,617	84	5.4
			120	759,529	608,516	44,091	80	5.8
				763,764	585,278	49,654	77	6.5
				801,862	635,247	42,700	79	5.3
	Gravel	6	72	859,681	759,547	42,516	88	4.9
				800,013	737,007	36,858	92	4.6
				1,095,577	911,062	47,231	83	4.3
			96	718,021	589,528	52,944	82	7.4
				994,725	562,589	62,258	87	6.3
				967,834	822,046	57,735	85	6.0
			120	816,686	642,626	74,379	79	9.1
				923,396	694,840	73,027	75	7.9
				743,990	651,777	82,420	88	11.1
75	Limestone	4	24	225,633	164,775	13,533	73	6.0
				278,559	190,149	15,681	68	5.6
				276,110	194,573	14,682	70	5.3
		5	16	333,954	227,770	8,686	68	2.6
				354,343	228,964	7,026	65	2.0
				363,955	241,026	7,744	66	2.1
			24	268,737	185,087	8,664	69	3.2
				297,224	203,519	9,252	68	3.1
				332,788	203,887	8,698	61	2.6
			32	327,789	226,518	12,019	69	3.7
				445,438	283,872	14,001	64	3.1
				315,230	222,273	12,368	71	3.9
			40	350,192	253,833	15,149	72	4.3
				370,765	257,155	13,979	69	3.8
				347,031	242,731	16,159	70	4.7
		6	16	491,430	278,349	3,661	57	0.7
				231,371	166,319	2,690	72	1.2
				221,417	154,068	2,878	70	1.3
				209,991	151,446	3,535	72	1.7
				228,653	166,013	3,280	73	1.4
			24	496,643	327,513	4,122	66	0.8
				348,515	237,710	4,675	68	1.3
				377,337	238,833	4,947	63	1.3
				249,782	180,470	4,782	72	1.9
				258,316	180,819	4,779	70	1.9
			32	273,928	190,844	6,099	70	2.2
				275,369	191,008	7,316	69	2.7
				259,534	181,324	6,628	70	2.6
				265,634	187,014	6,586	70	2.5
				260,466	182,303	6,827	70	2.6
			40	314,430	216,977	8,006	69	2.6
				302,369	204,796	8,234	68	2.7
				350,747	229,245	8,182	65	2.3
				315,328	208,147	8,582	66	2.7
				336,668	213,070	7,632	63	2.3

(Continued)

TABLE G-1. (Continued)

Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Instantaneous Resilient Modulus E_{RI}, psi	Total Resilient Modulus E_{RT}, psi	Modulus of Cumulative Total Deformation E_{TT}, psi	$\frac{E_{RT}}{E_{RI}}$, %	$\frac{E_{TT}}{E_{RI}}$, %
75	Limestone	7	16	627,581	300,146	2,089	48	0.3
				437,678	271,913	2,644	62	0.6
				515,331	346,493	2,914	67	0.6
				478,894	277,706	1,899	58	0.4
				243,089	173,424	1,785	71	0.7
				228,535	155,477	2,043	68	0.9
			24	474,578	287,084	4,674	60	1.0
				484,078	290,666	3,834	60	0.8
				498,184	299,203	3,866	60	0.8
			32	600,493	340,556	6,423	57	1.1
				564,354	302,202	4,494	54	0.8
				572,838	342,263	9,383	60	1.6
				419,517	277,999	5,598	66	1.3
			40	471,901	297,230	8,574	63	1.8
				439,142	274,439	7,672	62	1.8
				503,033	309,422	7,948	62	1.6
		8	16	384,721	231,131	1,252	60	0.3
				397,839	243,446	1,642	61	0.4
			24	387,531	245,770	2,188	63	0.6
				409,240	238,595	2,737	58	0.7
				424,313	259,822	2,991	61	0.7
				268,660	175,808	2,584	65	1.0
				285,766	186,990	2,442	65	0.9
	Gravel	4	24	386,204	264,945	13,381	69	3.5
				275,562	184,578	14,043	67	5.1
				286,392	193,639	13,791	68	4.8
		5	16	416,822	266,147	9,202	64	2.2
				413,454	268,272	9,608	65	2.3
				433,763	256,944	9,422	59	2.2
			24	565,794	354,503	9,648	63	1.7
				343,635	234,089	11,774	68	3.4
				299,985	222,757	11,497	74	3.8
			32	411,406	268,781	15,023	65	3.7
				455,584	297,393	17,119	65	3.8
				400,227	259,898	14,520	65	3.6
			40	453,784	279,480	17,023	62	3.8
				391,393	276,570	17,581	71	4.5
				454,786	282,669	17,957	62	4.0
		6	16	517,796	334,967	9,554	65	1.9
				236,244	178,593	8,647	76	3.7
				233,259	178,314	8,094	76	3.5
				240,207	185,697	8,437	77	3.5
			24	592,429	382,123	11,857	65	2.0
				381,580	271,418	9,932	71	2.6
				383,359	264,526	9,099	69	2.4
				274,007	197,613	10,377	72	3.8
				269,746	196,280	9,610	73	3.6
			32	142,810	89,282	12,646	63	8.9
				298,285	219,291	12,954	74	4.3
				342,147	244,188	13,253	71	3.9
				305,761	225,107	11,703	74	3.8
				304,177	226,748	12,945	75	4.3

(Continued)

TABLE G-1. (Continued)

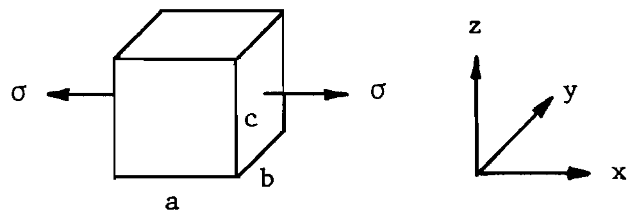
Temperature, °F	Aggregate	Asphalt Content, %	Stress Level, psi	Instantaneous Resilient Modulus E_{RI}, psi	Total Resilient Modulus E_{RT}, psi	Modulus of Cumulative Total Deformation E_{TT}, psi	$\frac{E_{RT}}{E_{RI}}$, %	$\frac{E_{TT}}{E_{RI}}$, %
				354,173	256,314	17,553	72	5.0
				337,943	233,303	17,890	69	5.3
		6	40	345,254	240,114	16,380	70	4.7
				330,497	228,231	15,689	69	4.8
				348,025	243,790	17,032	70	4.9
				479,207	296,718	3,475	62	0.7
				506,915	292,649	4,522	58	0.9
			16	517,553	298,337	3,755	58	0.7
				527,499	359,701	3,949	68	0.8
				255,733	185,994	5,715	73	2.2
				260,381	184,090	3,907	71	1.5
				480,132	293,932	4,938	61	1.0
				543,381	331,197	6,579	61	1.2
				494,346	302,449	5,625	61	1.1
75	Gravel	7	24	278,818	198,160	6,685	71	2.4
				299,022	208,150	5,967	70	2.0
				294,660	207,543	7,461	70	2.5
				303,530	207,787	6,085	68	2.0
				542,485	294,643	6,893	54	1.3
				297,262	215,037	11,057	72	3.7
			32	308,011	220,930	8,012	72	2.6
				296,690	206,861	6,978	70	2.4
				316,516	226,926	8,595	72	2.7
				376,934	260,003	11,510	69	3.1
			40	374,885	245,203	10,887	65	2.9
				393,736	255,543	11,227	65	2.9
			16	431,874	259,968	2,080	60	0.5
				410,474	252,247	2,952	61	0.7
		8	24	341,828	222,788	3,017	65	0.9
				345,099	223,765	3,121	65	0.9
				122,765	83,076	1,383	68	1.1
			8	129,658	90,959	1,125	70	0.9
				125,558	98,403	1,219	78	1.0
				164,102	118,535	3,329	72	2.0
	Limestone	6	16	155,207	105,868	3,019	68	1.9
				144,063	101,626	2,727	71	1.9
				180,214	126,726	5,393	70	3.0
			24	227,706	160,119	5,252	70	2.3
				216,761	140,309	5,555	65	2.6
100				177,655	134,160	4,192	76	2.4
			8	170,252	124,077	4,220	73	2.5
				160,148	114,944	3,987	72	2.5
				221,902	158,366	7,316	71	3.3
	Gravel	6	16	202,251	144,197	6,880	71	3.4
				215,481	150,430	6,585	70	3.1
				228,751	168,531	9,621	74	4.2
			24	220,263	172,136	10,516	78	4.8
				208,309	162,580	9,334	78	4.5

APPENDIX H

DERIVATION OF UPPER LIMIT FOR POISSON'S RATIO

APPENDIX H. DERIVATION OF UPPER LIMIT FOR POISSON'S RATIO

ELEMENT IN TENSION (Ref 76)



Consider a cube of sides a, b, and c in the x, y, and z directions, respectively, and assume that tensile stresses of intensity σ are applied on the faces perpendicular to the x axis. The initial volume of the cube is $Vo = abc$. Since strain $\varepsilon = \frac{\sigma}{E}$, the final volume V is found to be

$$V^1 = a(1 + \varepsilon) \times b(1 - \nu\varepsilon) \times c(1 - \nu\varepsilon) = Vo(1 + \varepsilon)(1 - \nu\varepsilon)^2$$

By neglecting powers of ε higher than 1, then

$$V^1 = Vo + Vo\varepsilon(1 - 2\nu)$$

Increase in volume is

$$\Delta V = V^1 - Vo = Vo\varepsilon(1 - 2\nu)$$

and since

$$\Delta V \geq 0 \text{ and}$$
$$\varepsilon > 0,$$

it follows that

$$1 - 2\nu \geq 0$$

or

$$\nu \leq 1/2.$$

ELEMENT IN COMPRESSION

From the above, one can present the case for compression as follows:

$$V^1 = a(1 - \varepsilon) \times b(1 + \nu\varepsilon) \times c(1 + \nu\varepsilon) = Vo(1 - \varepsilon)(1 + \nu\varepsilon)^2$$

By similarly neglecting powers of ε higher than 1,

$$V^1 = Vo(1 - \varepsilon)(1 + 2\nu\varepsilon) = Vo + Vo\varepsilon(2\nu - 1)$$

Change in volume is

$$\Delta V = V^1 - Vo = Vo\varepsilon(2\nu - 1)$$

Since

$$\Delta V \leq 0 \text{ and}$$
$$\varepsilon > 0,$$

it follows that

$$2\nu - 1 \leq 0$$

or

$$\nu \leq 1/2.$$