NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT **39**

FACTORS INVOLVED IN THE DESIGN OF ASPHALTIC PAVEMENT SURFACES

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 39

FACTORS INVOLVED IN THE DESIGN OF ASPHALTIC PAVEMENT SURFACES

FRED N. FINN
MATERIALS RESEARCH AND DEVELOPMENT, INC.
OAKLAND, CALIFORNIA

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS IN COOPERATION WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
PAVEMENT DESIGN
BITUMINOUS MATERIALS AND MIXES

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1967

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

NCHRP Project 1-8 FY '65
NAS-NRC Publication 1543
Library of Congress Catalog Card Number: 67-61866

FOREWORD

By Staff

Highway Research Board

In recent years considerable effort has been devoted to exploring methods for improving performance of flexible pavements, with major emphasis on the base course layers. This report deals with asphaltic surface course materials, properties, tests, and design considerations, with particular attention to factors not generally used in current design procedures. It reviews a large volume of previous work in this field and provides a current state of the knowledge on the subject plus recommendations for future research. The report will be of special interest and value to research and bituminous materials engineers, as well as design personnel.

With the constantly increasing traffic volumes and loads, there is understandably a corresponding interest in the building of better roads. In the area of flexible pavements, attention in recent years has been focused on the designing of thicker sections and increasing the strength characteristics of base courses. Consequently, a need exists for evaluation of surface course design methods to ensure that they keep pace with developments in the design of other layers of the pavement. This involves the identification of the fundamental factors affecting mix design and the determination of the relationship between these factors and total pavement performance.

The research agency, Materials Research and Development, Inc., has made an extensive study of existing research literature to identify and evaluate the factors pertaining to asphaltic surface course materials and testing techniques. Emphasis was placed on an analysis of characteristics such as stiffness, fracture strength, thermal stresses, fatigue, and durability that are not generally included in current mix design procedures. The report contains a review of past investigations in these areas plus the researchers' interpretation of application of each to future design of surface courses. Discussion of measurement methods presently available for studying the various characteristics is also included in the report.

Inasmuch as the study reviewed factors not generally used in surface course design and did not involve appreciable laboratory investigations, this final report of the project is not intended to draw conclusions on new design methods. It does, however, contain recommendations for the direction of future research on the most likely factors that could be used in asphaltic surface course design. The researchers also suggest an experimental procedure for a study of the fatigue of asphaltic surfaces that could be used as a guide for a field study to correlate field performance of flexible pavements with the surface course characteristics reviewed in the report.

CONTENTS

FIGURES

FIGURES

FIGURES

TABLES

TABLES

ACKNOWLEDGMENTS

For the research agency, Materials Research and Development, Inc. (a Division of Woodward, Clyde, Sherard, and Associates), Fred N. Finn, Vice President, Engineering, acted as Principal Investigator, with Carl L. Monismith as consultant, and B. A. Vallerga as administrator.

Grateful acknowledgment is also extended to the following staff personnel of the research agency:

R. G. Hicks, for assistance with computations and analyses of fatigue data, supervision of the fatigue tests, and assistance in the review of various publications.

Richard White, for assistance in the preparation and testing of asphalts used in the fatigue tests, and for helpful suggestions relative to the properties of asphalt pertinent to this study.

C. J. Van Til, for invaluable assistance in reviewing the various portions of the report.

FACTORS INVOLVED IN THE DESIGN OF ASPHALTIC PAVEMENT SURFACES

SUMMARY

The purpose of this report is to present a comprehensive review of information related to factors which influence the design of asphaltic surfaces in terms of loading conditions, strength requirements, and mix properties. The report is mainly concerned with factors which are not as yet included in mix design requirements for asphaltic surfaces. Those factors specifically included are (a) rheological properties, (b) fracture or tensile strength, (c) thermal stresses, (d) fatigue characteristics, and (e) durability or long-term performance. A brief review of such factors as (a) designs for standing loads, (b) designs for braking and acceleration, and (c) designs for orthotropic bridge decks is also included.

The technique used in preparing the report was basically to review a representative portion of available research literature, some of which has not received general distribution, and to analyze the information in an effort to obtain some consensus as regards the conclusions reached by various investigators. In preparing the report, each chapter has been divided into two basic subdivisions. The first part is a review of available information dealing specifically with the individual materials (asphalt and aggregate) and the asphaltic mixture. The second part is an interpretation, by the authors of this report, dealing with applications to the design of asphaltic mixes. In some cases the interpretation deals with induced stresses and strength characteristics pertinent to asphaltic surfacings.

Investigations of this type do not lend themselves to specific findings but deal somewhat with the state of knowledge at a particular time, attempting to show where further work is required. Information reviewed for this study can be briefly summarized as follows:

1. The rheological or stiffness properties of asphalt and asphaltic concrete have been thoroughly investigated, both in the United States and Europe. Although these investigations have shown that asphalt and asphaltic concrete are both time-of-loading and temperature dependent, they can be expected to act elastically for specific conditions. For example, as long as the temperature and rate of loading do not vary markedly, a mixture of asphaltic concrete will act elastically up to approximately 0.1 percent strain. Thus, it is possible to analyze asphaltic mixtures according to the theory of elasticity for a given situation as represented by the modulus of elasticity or stiffness modulus. Methods have also been developed, or carried over from related technical disciplines, which make it possible to readily predict the stiffness modulus over a wide range of temperatures and times of loading from a comparatively limited number of tests. There is not complete unanimity as regards the applicability of treating asphaltic mixtures as elastic or linear-viscoelastic materials; however, there appears to be a strong consensus that this approach should be pursued as far as possible before attempting to deal with this material by more complicated methods.

2. Fracture strength of asphalt and asphaltic concrete has been evaluated at various temperatures and loading conditions. Tensile strengths of asphaltic concrete at

low temperatures have been reported at values ranging from 550 to 1,400 psi with strains of approximately 1 percent. Although this level of strain is excessive for large numbers of applications, it can be useful to designs for limited load applications and for interpretations related to available strength to resist thermal stresses. It is also pertinent that maximum tensile strengths are achieved with asphalt film thicknesses in excess of 10 microns. Because most asphaltic mixes are manufactured at film thicknesses less than 10 microns, some further thought should be given to increasing these film thicknesses.

3. Thermal stresses in asphaltic concrete can be calculated on the basis of a knowledge of the thermal properties of an asphalt-aggregate system and temperature conditions. This is one of the main points stressed in the limited research dealing with this subject. Although such computations are rather involved, they appear to be reliable and should be useful. Thermal stresses, in most cases, do not approach the expected tensile strength of properly constructed asphaltic concrete. However, in extreme climatic conditions, possible in certain areas of the United States, it is conceivable that thermal stresses could exceed tensile strength. Also, and more pertinent, is the effect of thermal stresses and live load which, when combined, may significantly reduce fatigue life.

4. Fatigue properties represent a substantial portion of the report. This was considered one of the more important aspects of the investigation because it combines asphaltic concrete properties with the structural design requirements of an asphalt-type pavement. Test procedures are described in the report which, when combined with the multi-layered theory for computing stress and strain in the asphaltic surfacing, can be used at least qualitatively to predict expected performance. The stiffness modulus is shown to be a good determinant of fatigue life. Also, the interpretive portion of this chapter suggests that for asphaltic surfacings greater than 4 in. in thickness, it may be desirable to use asphalt cements of low penetration or high viscosity. For asphaltic surfaces less than 3 in. thick, the high-penetration or low-viscosity asphalts appear to be preferred. Investigations have also shown the importance of achieving low (3 to 5 percent) void contents for good fatigue performance.

5. Durability, particularly of asphalt, has been the subject of extended research since around 1897. A tremendous amount of laboratory information has been obtained, and there appears to be a definite correlation established between asphalt consistency and long-term performance. There is, however, a lack of definition as to the means by which asphalt performance has been evaluated in the field, hence there is a considerable difference of opinion among researchers as to the real significance of durability tests. If future research is to be more meaningful than past research, it will be essential to develop guidelines for evaluating the performance of asphalt in in-service pavements.

6. Requirements for the design of asphaltic surfacings subjected to uniquely high stresses due to standing loads, acceleration, or deceleration are discussed. Although research is limited in these areas, it appears that sufficient information is available from which special designs can be developed. The requirements for orthotropic bridge decks appear to be in the experimental stages. Experience in Europe has been relied upon for the design of some surface systems in the United States, and these are discussed.

It was originally hoped that some information would be available for the design of asphaltic concrete to be placed over cracked surfaces. Research to reduce crack reflection has generally concentrated on (a) methods for developing bond-breaking layers between the old surface and the new surface or (b) wire reinforcement in the new surface. Because of the lack of available information, this item has not been included in the report.

Methods of testing are discussed in the report. Because most of the factors are still in the research phase, little standardization of test methods has been attempted. However, tests are available for each factor discussed; therefore, the engineer is provided with the tools necessary for evaluation.

CHAPTER ONE

INTRODUCTION

In the design of asphaltic paving mixtures for highway pavements, it is generally agreed that a balance must be obtained between a number of desirable mix properties. Generally speaking, this balance necessitates a compromise in the selection of the final design asphalt content. To illustrate this point, the pertinent mix properties, at least those which appear important to the authors of this report, are discussed briefly. These properties may be enumerated as:

1. Stability.
2. Durability.
3. Flexibility.
4. Fatigue resistance.
5. Skid resistance.
6. Permeability or imperviousness.
7. Fracture (tensile) strength.

From a construction standpoint, the workability characteristics of the mix must also be accounted for. However, in this discussion it is assumed that the type of mixes to be discussed can be placed so long as proper construction procedures are followed; hence, workability considerations are included. This is not meant to imply, however, that this factor is not important, because proper placement of any asphaltic paving mixture is required so that each of the previously noted characteristics is optimized, thus assuring maximum life for the paving mixture in the particular environment in which it is placed.

In addition to the foregoing characteristics, the rheologic properties of a mixture (i.e., the stiffness or stress-strain characteristics of the mixture as a function of time of loading and temperature) must be evaluated. As will be seen in subsequent sections, this characteristic exerts an influence on other mix properties, such as fatigue resistance.

Under certain circumstances, the thermal characteristics of the mixture must also be evaluated. This is particularly true where the asphaltic mixture is subjected to environments in which very cold temperatures are highly probable.

In this chapter, then, a brief resumé of the pertinent mix properties is presented, together with an indication of the influence of asphalt content on each, to emphasize that the selection of the design asphalt content is an adjustment to optimize all of the desirable mix properties for a particular set of circumstances.

Stability

In the classical sense, stability has been defined as the resistance of a mix to deformation under load. The deformation implied in this definition is permanent or plastic deformation resulting from (a) slowly applied loads at relatively high temperatures, or (b) rutting associated with many applications of channelized traffic. Hveem and Vallerga have well summarized the factors influencing stability, and a modified version of their analysis is included in Figure 1, from which it will be noted that stability is a function of (a) frictional resistance, both interparticle and intraparticle (mass viscosity), (b) cohesion, and (c) inertia. Of these, frictional resistance is a major contributor to resistance to deformation; and for the high temperatures and slowly applied loads normally considered, the contribution of interparticle friction to stability is predominant. For these circumstances, the aggregate characteristics, particularly particle surface texture, exert a major influence. Improper compaction or high asphalt contents tend to reduce this friction and thus permit plastic deformations to develop more readily.

From this analysis, it can be seen that to promote sufficient resistance to deformation for a particular level of traffic, the asphalt content must be maintained at a comparatively low value so that the frictional resistance of the aggregate mass will be maintained at the level necessary to carry the load, assuming, of course, that the aggregate has sufficient frictional resistance and the mix has been properly compacted.

Durability

Durability of a paving mixture can be defined as its resistance to weathering, including aging, and to the abrasive action of traffic. Durability also refers to the ability of materials to resist change through weathering. Included

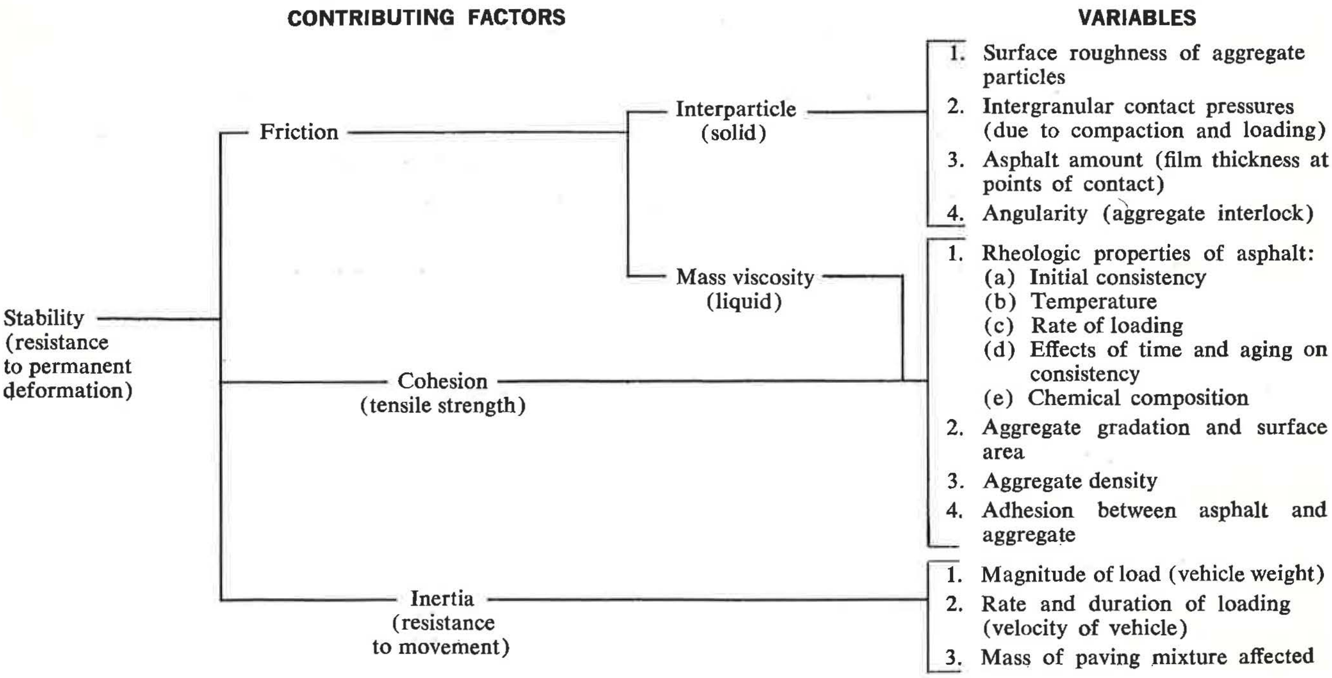


Figure 1. *Analysis of stability of asphaltic paving mixtures (after Hveem and Vallerga).*

with the effects of weathering are (a) changes in the characteristics of the asphalt due to such causes as volatilization, oxidation, polymerization, separation, and syneresis, and (b) changes in the mixture due to the action of water and water vapor. To minimize the effects of weathering, experience would indicate that high asphalt contents, dense gradations of aggregate, and well-compacted impervious mixtures are required.

Sufficient asphalt must also be incorporated in the mix to provide tensile properties adequate to resist the tractive or abrasive forces of traffic. In addition, the durability characteristics of the aggregate are important, in that the material must offer sufficient resistance to fracture and degradation under the forces imposed during the construction process and when the mixture is subjected to traffic.

From this brief discussion it can be seen that, with all other factors constant, good durability characteristics in a mixture are promoted by as high an asphalt content as possible. Because of this requirement, an anomaly exists in that high stability is associated with comparatively lower asphalt contents.

Flexibility

In this report, flexibility is defined only as the ability of the mixture to conform to long-term variations in base and subgrade elevations. Other definitions of flexibility embrace fatigue resistance as well as this long-term flexibility (*97a*). Examples of these long-term settlements which must be accommodated are those resulting from consolidation of underlying soft, compressible soils and differential compaction from traffic of the underlying components of the pavement structure.

Many pavements must be constructed on fills resting on soft, compressible clay deposits, and in many instances these deposits are of nonuniform thickness. Differential settlements will thus occur in the fill structure and be reflected in undulating pavement surfaces. The asphaltic mixture should have the ability to conform to the differential settlements which are long term, otherwise cracking will occur. In addition, it is virtually impossible to develop completely uniform compression characteristics in bases, subbases, and compacted subgrades at the time of construction. Thus, localized sections of the pavement structure will sometimes tend to compact differentially. Mixtures must also have the ability to conform to these localized settlements.

Although the data in this area are meager, experience would seem to indicate that flexibility is promoted by high asphalt contents and comparatively open (as opposed to dense) gradations of aggregate.

Fatigue Resistance

Heavy-duty pavements subjected to many repetitions of load may exhibit cracking of the asphaltic concrete resulting primarily from the resilient or elastic deformations to which the material is subjected. Moreover, in its initial phases this cracking may be associated with little or no permanent (or irrecoverable) deformation of the pavement section. That this type of failure occurs and that it is associated with load repetitions has been well documented by Hveem (*66*). When examining this particular mix property, however, one must consider the behavior of the entire structural pavement section because the occurrence of fatigue cracking is not only influenced by the characteristics of the asphaltic mixture, but also by the thickness and characteristics of the other components comprising the pavement section.

Available data seem to indicate that all mixtures exhibit fatigue and that the initiation of fatigue cracking is related either to the intensity of the tensile stress or strain induced in the mixture, with a high stress or strain associated with a lesser number of repetitions to the onset of cracking. Thus, the intensity of load, the thickness of the surfacing and its stiffness, and the thickness and resilient characteristics of the underlying materials all must be evaluated in order to determine the stress or strain to which the mixture will be subjected.

Although fatigue under repetitive stress is an established fact for asphaltic mixtures, it appears that mix variables influence the number of repetitions to failure at either controlled stress or strain loading. As is noted in a subsequent chapter, however, the influence of these variables on the fatigue resistance of mixtures is confounded by the mode of loading to which they are subjected. For example, if mixtures are subjected to a controlled stress mode of loading, those factors which tend to increase mixture stiffness, such as high asphalt viscosity and dense aggregate gradations, increase the service or fracture life at a particular stress level. On the other hand, in a controlled strain mode of loading, the lower the asphalt viscosity, the longer the fatigue life. It is hoped that some perspective will be placed on this anomalous behavior in the report.

Regardless of the method of testing, however, asphalt content and mixture density appear to have a clearly established influence, with a longer life associated both with higher asphalt content and increased density.

Skid Resistance

Skid resistance is the ability of the surface of an asphaltic paving mixture to provide sufficient friction so that a vehicle will be able to brake to a stop within a reasonable distance under a variety of environmental conditions. As with the property of fatigue resistance, other factors, in addition to mixture variables, must be considered. Chief among these are surface contaminations, such as oil films from crankcase drippings, excessive films of water (which may permit hydroplaning of tires), and snow and ice. These, however, are more of a temporary nature, and the design engineer, although recognizing these factors, must insure that the mixture itself will embody sufficient skid resistance so that for the majority of the time when the tire and pavement are in contact adequate skid resistance will be available.

High skid resistance is generally promoted by the same factors which contribute to high stability; that is, comparatively low asphalt contents and aggregates with rough surface textures. Suitable aggregates, in addition to possessing rough textures, must also be resistant to the polishing action of traffic. Aggregates most desirable from this standpoint are those which have minerals of different wear characteristics. Under the action of traffic, an aggregate of this type will continually have its rough surface texture renewed, thus providing the sharp coarse-grained surface necessary to develop effective contact of the tire and the pavement.

The amount of asphalt is also of primary importance. If an excess is present for the particular mix and environmental conditions, it is probable that bleeding of the asphalt to the pavement surface will result. This, in turn, will reduce the frictional resistance of the surface under wet weather conditions.

Permeability

Permeability of a mixture of asphalt and aggregate can be simply defined as the ease with which air, water, and water vapor will pass into or through the mixture. For highway pavements, it appears that mixtures with a high degree of imperviousness to air, water, and water vapor are desirable to promote long-term durability and to allow surface water to be transported to drainage facilities rather than to percolate through the pavement to the underlying components. For these conditions, then, the same factors which contribute to durability (i.e., high asphalt content, dense aggregate gradation, and "good" compaction) also insure imperviousness.

Under certain circumstances, a permeable mixture may be desired. This is normally accomplished by using a comparatively "open" or uniform gradation of aggregate. Methods for measuring laboratory and field permeability for construction control of density and for evaluation of mix durability have been suggested (*33, 47, 75*).

Fracture Strength

In this discussion, fracture strength is considered to be the maximum strength which a mixture exhibits when subjected to tensile forces. This strength is dependent on both rate of loading and temperature, being generally higher at more rapid rates of loading and lower temperatures. Fracture strength is important when considering the application of heavy loads to pavements, particularly at low temperatures and when the underlying pavement components are comparatively weak, such as in the spring of the year in many parts of the United States. In addition, the fracture strength of the mix is important when evaluating the possibility of cracking of the asphaltic mixture due to volume changes such as those which may result from temperature changes, aggregate absorption, and volume changes in the underlying components of the pavement structure.

Some of the mix variables which influence fracture strength are asphalt content; aggregate gradation, particularly the proportion of fine material (filler); type or mineralogical composition of fine material; and mixture density. Within limits, as the asphalt content and the proportion of fine material are increased, the fracture strength is also increased. Some evidence is also available that filler (i.e., hydrated lime) which appears to have more active surfaces in the presence of asphalt tends to give higher fracture strengths. In addition, as the density of the mixture is increased, particularly the degree of packing of the aggregate, the fracture strength is also increased.

From the foregoing discussion, certain mix variables reoccur in the majority of the mix properties which must be evaluated for proper design. These are asphalt content, aggregate gradation, and mix density (degree of compac-

TABLE 1

DESIRABLE CHARACTERISTICS TO OPTIMIZE MIXTURE PROPERTIES

MIX PROPERTY	ASPHALT CONTENT	AGGREGATE GRADATION	DEGREE OF COMPACTION
Stability	Low	Dense	High
Durability	High	Dense	High
Flexibility	High	Open	—
Fatigue resistance	High	Dense [a]	High
Skid resistance	Low	Dense or open [b]	High [c]
Imperviousness	High	Dense	High
Fracture strength	High	Dense	High

[a] Assuming a heavy-duty, comparatively thick layer of asphaltic concrete.

[b] Both types of gradations have indicated good skid resistance characteristics. What appears to be more important is the texture of the aggregate particles.

[c] Although compaction is not normally indicated for this property, it is implied to insure that aggregate particles will not dislodge under the tractive forces applied to the surface.

tion). Table 1, which presents the influence of these variables on a comparative basis, indicates that an asphalt content that attempts to strike a balance between all of the desirable mix properties must be selected. In addition, for the majority of the mix properties, dense gradations appear desirable, and proper compaction in the field is emphasized.

Although this summary has attempted to define briefly the important mix properties which must be considered in design and to illustrate that the design of mixtures for specific conditions requires a compromise between a number of mix variables, in particular asphalt content, its major purpose has been to set the stage for the ensuing portions of the report, which are concerned in detail with specific mix properties to which greater future attention must be given so that asphaltic concrete pavements can be better designed to withstand the demands of ever increasing traffic. These include fatigue resistance, durability, fracture strength, thermal properties (characteristics), and rheological properties.

In addition, some discussion is presented with respect to the behavior of mixes under both standing loads and shearing loads associated with braking and acceleration. As a part of this discussion, some information pertaining to the stability of mixtures is presented. However, less emphasis is placed on stability in this discussion as compared with the previously noted mix properties. This is not meant to imply that stability is not an important consideration. Available evidence indicates that adequate mixture stability is provided in the majority of situations. It is thus believed that engineers have sufficient understanding of this property to permit in this report more emphasis in other areas where confusion may exist and to which further research efforts should be directed.

Results of the AASHO Road Test indicated the loss of serviceability associated with rutting in the wheelpaths of asphaltic surfacings. Subsequent field studies reported by Rogers et al. (*127*) have indicated that this is not a general problem in the United States, and hence it is concluded herein that rutting is not a serious problem for well designed and constructed surfacings. Speers (*150*) has studied rutting in some detail and has attempted to use this parameter in laboratory test tracks as a means for evaluating the performance of asphaltic pavement structures and materials. This work has also provided significant information about parameters affecting rutting when this factor has posed a performance problem.

Because of recent developments in the design of bridge structures, such as the use of orthotropic bridge decks, some consideration is also given in the report to special mix requirements for bridge decks.

Hopefully, then, this report attempts (a) to discuss in detail the existing information on the previously noted properties, (b) to appraise this information relative to mixture performance, and (c) to identify areas where additional effort should be concentrated and the priority which should be followed in developing needed information. In addition, as a part of this evaluation, methods of measurement of these characteristics are discussed and general suggestions are included for the use of existing tests to measure these properties.

CHAPTER TWO

STIFFNESS OR RHEOLOGICAL CHARACTERISTICS

Stiffness, as used in this report, is the relationship between stress and strain as a function of time of loading and temperature; this relationship between stress, strain, and time is also referred to as the rheologic behavior of mixtures. In many applications of asphaltic concrete, its stiffness characteristics must be known not only to assess the behavior of the mix itself, but also to evaluate the performance of an engineering structure of which the mix is a part, such as a highway or airfield pavement.

Figure 2 shows a simplified diagram illustrating the time-of-loading dependence of the stiffness of asphaltic concrete for a particular temperature. At very short loading times, it will be noted that stiffness is essentially time independent; in this case, the stiffness approaches the elastic modulus. For an intermediate range on the time scale, the stiffness decreases with an increase in time of loading. At very long loading times, the stiffness may still decrease, but at a uniform rate, and the behavior may be considered to be purely viscous. The stiffness, under these circumstances, gives a measure of the flow characteristics of the mixture. (In Figure 2, this viscous deformation has been characterized by viscous traction to indicate behavior under axial stress; as noted in the figure, this viscous traction is approximately three times the viscosity, which is a measure of flow resulting from shear stresses.)

In general, the response to a three-dimensional system of stress, such as that which occurs in an asphaltic concrete layer of a highway pavement subjected to load, is fairly complex. In simple, isotropic, linearly elastic materials, two parameters (moduli) must be known. When the response depends on the time or rate of loading and the temperature, such as is the case for asphaltic mixtures, the characterization is even more difficult. Attention is directed, in this chapter, to investigations which have attempted to define this relationship between stress and strain.

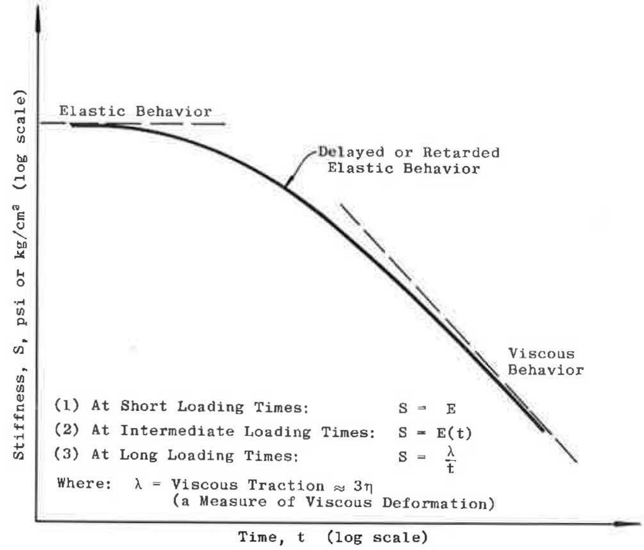


Figure 2. Idealized time-of-loading dependence of the stiffness (stress/strain) characteristics of an asphaltic material subjected to an axial tensile stress.

STIFFNESS ACCORDING TO VAN DER POEL

Van der Poel (*156, 157*) recommended that attention be concentrated on a single stress and its resultant strain, because for many purposes this is reasonably adequate. He has suggested a single parameter termed the stiffness, *S*, such that

$$\sigma = S(t,T)\ \epsilon \tag{1a}$$

or

$$S(t,T) = \sigma/\epsilon \tag{1b}$$

in which

S = stiffness, in psi or kg/cm²;
σ = axial stress;
ϵ = axial strain;
t = time of loading; and
T = temperature.

At very short times of loading and/or low temperatures, the behavior of asphaltic concrete is almost elastic in the classical sense, and the stiffness, *S*, is analogous to an elastic modulus, *E* (e.g., Fig. 2). At longer times of loading and higher temperatures, the stiffness is simply a relation between the applied stress and the resulting strain. Conversely, if the stiffness corresponding to a particular time and temperature and either the stress or the strain are known, an estimate can be made of the strain or the stress developed in the asphaltic mixture.

From an engineering standpoint, there appears to be justification for use of simple axial loading to determine stiffness, because variations in temperature and time of loading for the working range of asphaltic concrete result in a range of stiffnesses from approximately 10^6 to 10^9 psi, a factor of 1,000. This difference is very large in comparison to the difference in values obtained in compression and shear, or between the response in the lateral and longitudinal directions, a factor of, at the most, 3.

From both creep and dynamic tests, van der Poel developed data indicating that, in the case of mixtures containing dense-graded aggregates and asphalt cements and which were well compacted (approximately 3 to 5 percent air voids), the stiffness of a mixture is dependent on the stiffness of the asphalt which it contains and the volume concentration, C_v,* of the aggregate. Thus, with a knowledge of the penetration and softening point (ring and ball) of the asphalt as it exists in the mixture,† and the volume concentration of the aggregate, an estimate of the stiffness of asphaltic concrete can be determined from Figures 3 and 4.

Figure 3 permits determination of the stiffness of asphalt for a particular time of loading and temperature from the penetration and ring-and-ball softening point of the recovered asphalt. With the stiffness of the asphalt known, the stiffness of the mixture can be determined from Figure 4 using the volume concentration of the aggregate.

More recently, Heukelom and Klomp (*51*) have examined van der Poel's method in more detail and have suggested that Figure 6 gives a reasonable estimate of the stiffness of asphaltic concrete provided the stiffness of the asphalt is determined from Figure 5,‡ a modification of van der Poel's curve (Fig. 3). Figure 6 is based on

* The volume concentration is defined as

$$C_v = \frac{\text{Volume of compacted aggregate}}{\text{Volume of aggregate + asphalt}}$$

† Can be ascertained by extracting and recovering the asphalt from the mixture.

‡ Figure 5 is a modification of Figure 3 in that the stiffness is determined directly in kg/cm² rather than in newtons per square inch; in addition, the lines for negative penetration indices in Figure 5 are in a different location than in Figure 3.

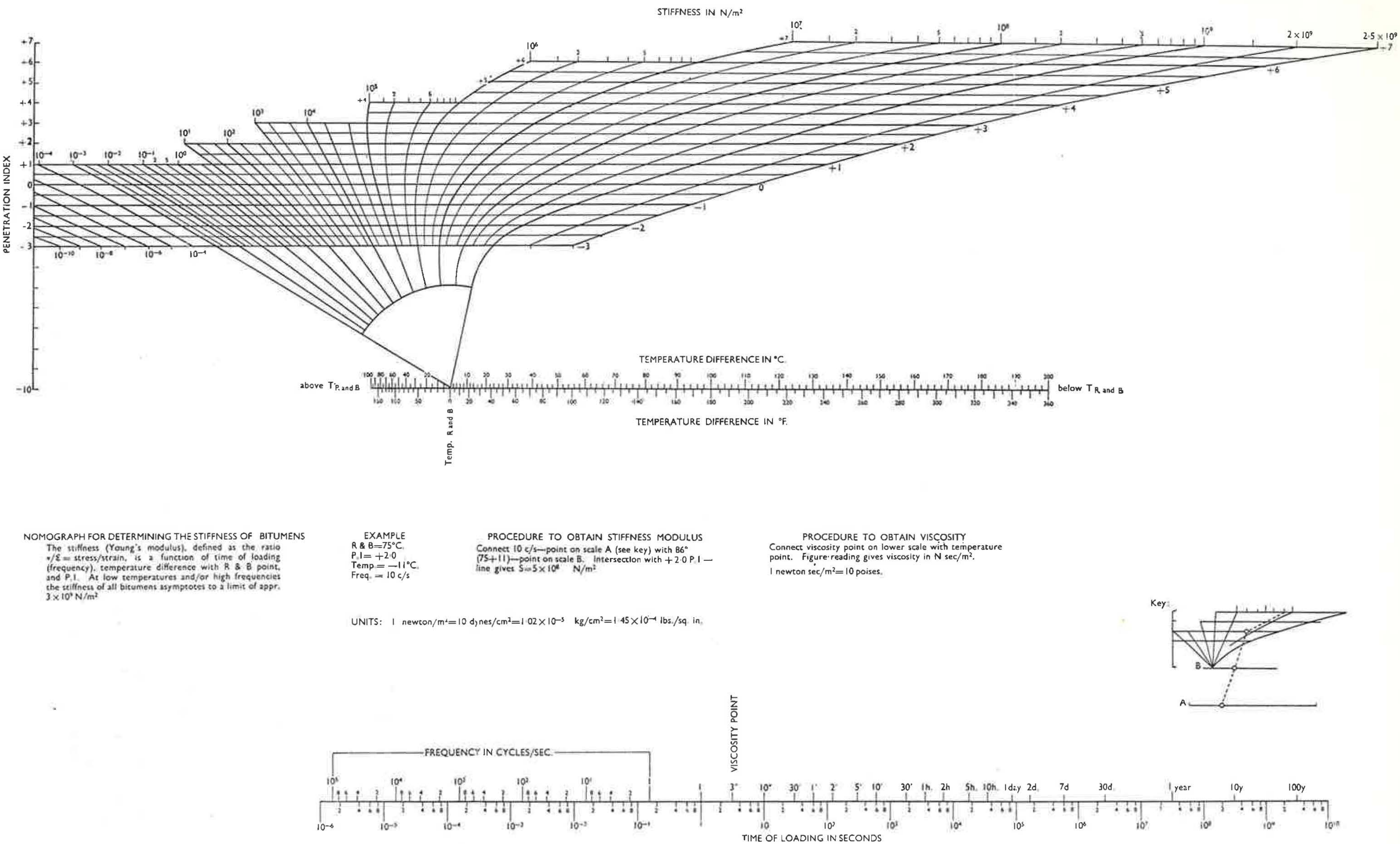


Figure 3. Nomograph for determining the stiffness of bitumens (after van der Poel, 157).

$$\frac{S_{mix}}{S_{bit}} = \left(1 + \frac{2.5}{n}\ \frac{C_v}{1 - C_v}\right)^n \tag{2}$$

in which

$n = 0.83 \log \dfrac{4 \times 10^5}{S_{bit}}$;

S_{mix} = stiffness of the mix, in kg/cm²; and

S_{bit} = stiffness of the asphalt, in kg/cm² (determined from Fig. 5).

The terms are applicable to well-compacted mixtures with about 3 percent air voids and C_v values ranging from 0.7 to 0.9.

Pell and McCarthy (*121*) have presented data which suggest that van der Poel's nomographs (Figs. 3 and 4) may be somewhat inaccurate at very small strain amplitudes and high temperatures based on stiffness measurements of asphalts and asphaltic mixtures during the conduct of fatigue tests. However, for the majority of cases which they investigated, computed stiffnesses compared reasonably well with measured stiffness values, as shown in Figure 7. It should be noted that these values are within the accuracy which van der Poel attributes to the nomograph (Figure 3) for stiffness determinations on the asphalt; i.e., a factor of two.

In another investigation, Monismith (*101*) has also checked laboratory-determined stiffness values on both laboratory-compacted specimens of asphaltic concrete and specimens saved from asphaltic concrete pavements. These data are summarized in Table 2, in which it will be noted that the stiffnesses for dense-graded mixtures computed from the nomograph agree reasonably well with those measured on 1.5 × 1.5 × 15-in. beams when the air void content of the mixtures is in the range 3 to 4 percent. For specimens with void contents greater than this, the predicted stiffness, as would be expected, is higher. The agreement for the base course specimens is poor. This may be due in part to the fact that the stiffness measurement was affected by the size of the specimen, which unfortunately was comparable with that of the largest aggregate. In addition, the asphalt content for these specimens was lower than that which would normally be used for this aggregate as a surface course, which in turn is outside the limits of the original correlation.

The range in measured stiffnesses as shown in Table 2 is, at least in part, due to the normal variations which can be expected from random samples and, in part, due to a variation in densities of the specimens. In general, it appears, however, that this approach holds promise as being an expedient method for stiffness determination, provided the mixtures under consideration fall within the range for which the nomograph was developed—that is, well-compacted, dense-graded, surface course mixtures.

RHEOLOGIC CONSIDERATIONS

Other investigators have suggested more direct procedures for measuring the time and temperature dependence of specific mixtures by direct testing in compression, tension, and flexure. This discussion also is limited to uniaxial stress-strain behavior. Subsequently, however, some information is presented to illustrate the extension of one-dimensional

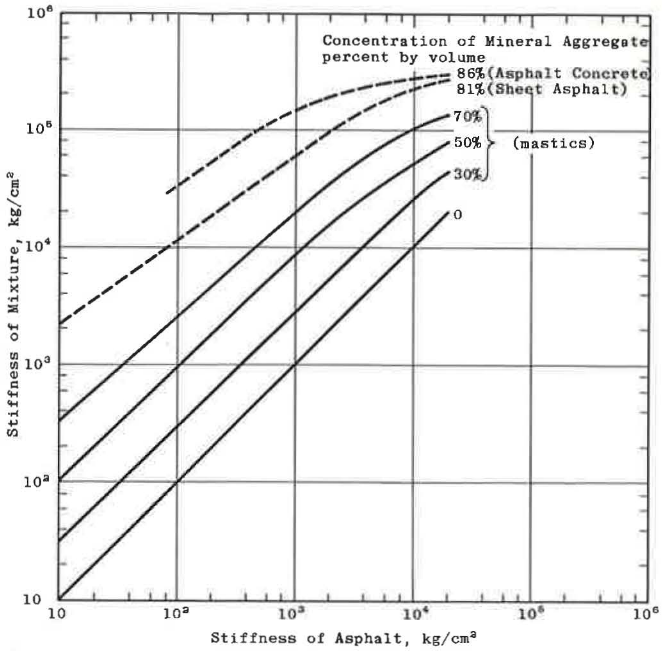


Figure 4. Relation between stiffness of asphalt and stiffness of mixtures (after van der Poel, 157).

behavior to three-dimensional response such as that which occurs in pavements.

The types of tests have included the following:

1. Creep.
2. Stress relaxation.
3. Constant rate of strain.
4. Dynamic:
   (a) Sinusoidal variation of stress or strain with time.
   (b) Step function pulse loading where the duration of the pulse (usually load) corresponds to the velocity of a vehicle, termed repeated loading.

These types of loading, together with measured responses, are shown schematically in Figure 8.

In *creep* tests, which have been performed on asphaltic mixtures in compression, tension, and flexure, a constant stress is applied and strain is measured as a function of time (Fig. 8a). Normally this test, as others described, is conducted at constant temperature for asphaltic mixtures. From the creep test results, a measure of stiffness, termed *compliance*, can be determined.

$$D(t) = \frac{\epsilon(t)}{\sigma_0} \tag{3}$$

in which

$D(t)$* = compliance, in sq in./lb or sq. cm/kg;

$\epsilon(t)$ = measured strain as a function of time; and

σ_0 = constant stress.

* This symbol for compliance corresponds to that recommended by the Society of Rheology for defining the response in tension and compression.

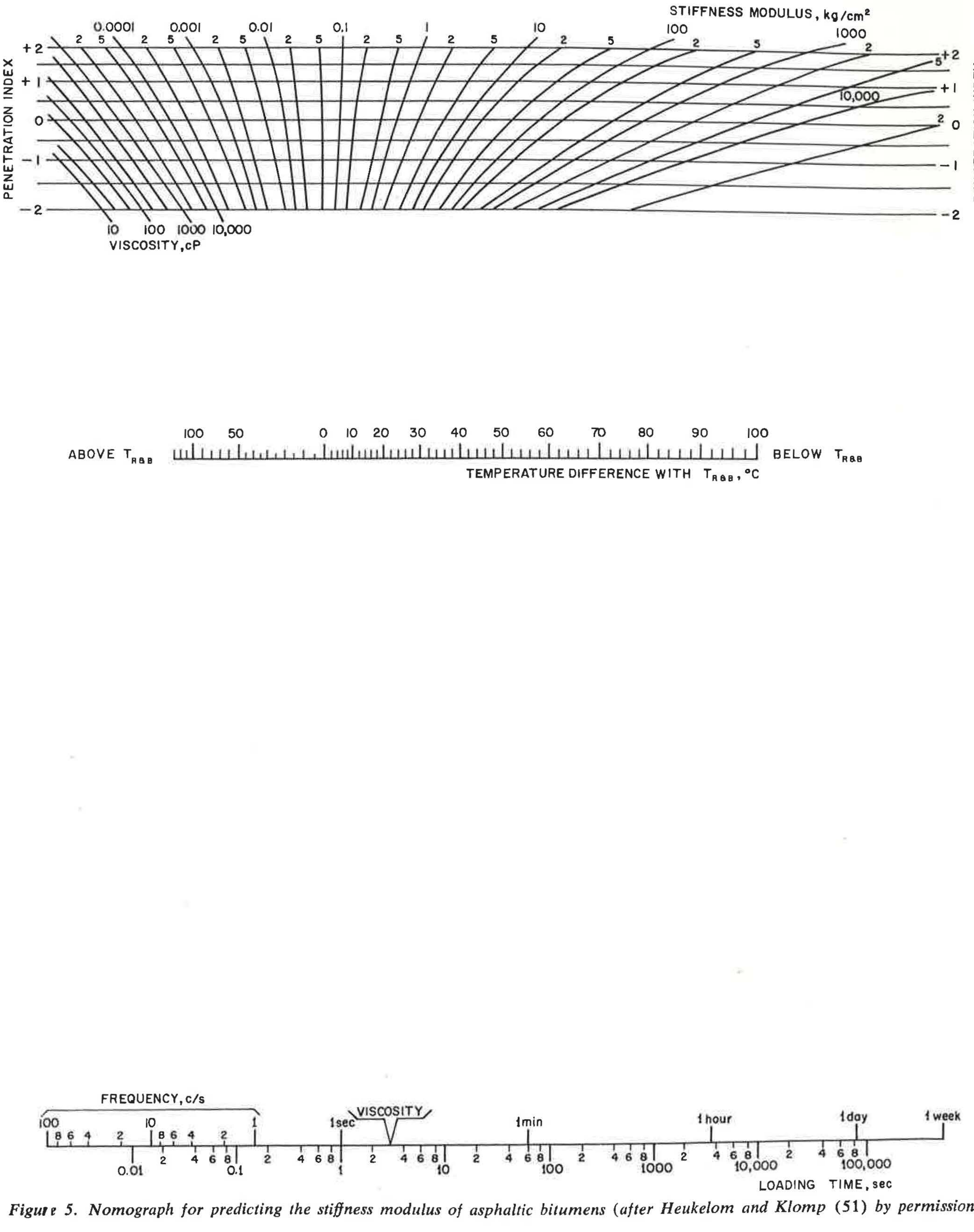


Figure 5. Nomograph for predicting the stiffness modulus of asphaltic bitumens (after Heukelom and Klomp (51) by permission).

In *stress relaxation* tests, a constant strain is applied and the resulting stress is measured as a function of time (Fig. 8b). From these results, the relaxation modulus, $E(t)$ (analogous to van der Poel's stiffness), can be determined from

$$E(t) = \frac{\sigma(t)}{\epsilon_0} \tag{4}$$

in which

$E(t)$ = relation modulus, in psi or kg/cm²;
$\sigma(t)$ = measured stress as a function of time; and
ϵ_0 = constant strain.

At a particular temperature the stiffness, S, determined from Eq. 1a is, for all practical purposes, the same as the modulus, $E(t)$, determined from Eq. 4. Assuming the material to be linearly viscoelastic, the relaxation modulus, $E(t)$, can also be obtained by numerical procedures from the creep compliance.

A measure of stiffness can also be determined from *constant rate-of-strain* tests (Fig. 8c). The relaxation modulus in this instance is computed from the slope of the stress-strain curve at a particular rate of strain, or

$$E(t) = d\sigma/d\epsilon \tag{5}$$

in which $d\sigma/d\epsilon$ is the slope of the stress-strain curve at a particular strain rate. By performing this type of test at different rates of strain, the modulus can be obtained as a function of time of loading. If the material exhibits simple viscoelastic behavior, the modulus determined from Eq. 5 should be the same as that determined from Eq. 4 as a function of time.

Another approach to the determination of response as a function of time is through the application of *sinusoidal loading* to a specimen (Fig. 8d). If the material is viscoelastic, the deformation resulting from the load will have the same sinusoidal variation with time but will lag behind the stress by a time represented by ϕ/ω, where ϕ is the phase angle (or phase shift) between the stress and the resulting strain and ω is the frequency of loading. From the peak amplitudes of stress and strain, a modulus, E^*, can be determined:*

$$|E^*| = \frac{\sigma_0}{\epsilon_0} \tag{6}$$

* As noted, the complex modulus is dependent on frequency. From an engineering standpoint, the absolute value of the complex modulus defined by Eq. 6 at a particular frequency is the measure of stiffness useful to the solution of structural problems (*22*). From a rheologic point of view, both the absolute value of the complex modulus and the phase shift are necessary to define the response of the material. The phase shift, ϕ, is essentially a measure of the deviation of the material from purely elastic behavior. For example, if the material were linearly elastic, ϕ would be zero. As the material deviates from purely elastic behavior, the phase shift, ϕ, will increase, particularly at lower frequencies (e.g., Fig. 20). Considering again the relationships shown in Figure 8d, if a stress which varies according to the relation $\sigma = \sigma_0 \sin \omega t$ is applied to a specimen, the resulting strain will also vary sinusoidally with time but will be displaced by the phase angle according to the relation $\epsilon = \epsilon_0 \sin(\omega t - \phi)$. From these equations, the relation between σ, ϵ, E^*, ϕ, and ω can be developed and has the form

$$\omega = \sigma_0 \, |E^*| \, [\sin \omega t \cos \phi - \cos \omega t \sin \phi] \tag{6a}$$

in which

$|E^*|$ = absolute value of complex modulus at a particular frequency of loading;
σ_0 = peak amplitude of sinusoidal stress at corresponding frequency; and
ϵ_0 = peak amplitude of sinusoidal strain at same frequency.

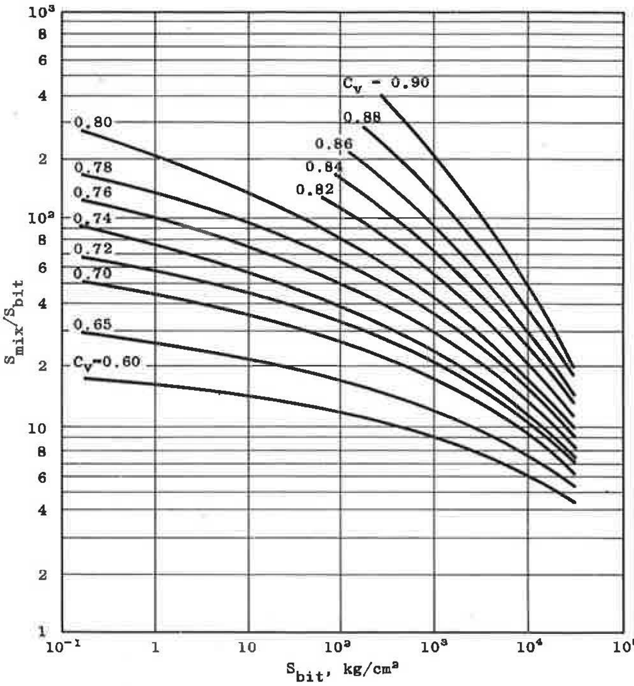


Figure 6. S_{mix}/S_{bit} as a function of S_{bit} and C_v (after Heukelom and Klomp (51) by permission).

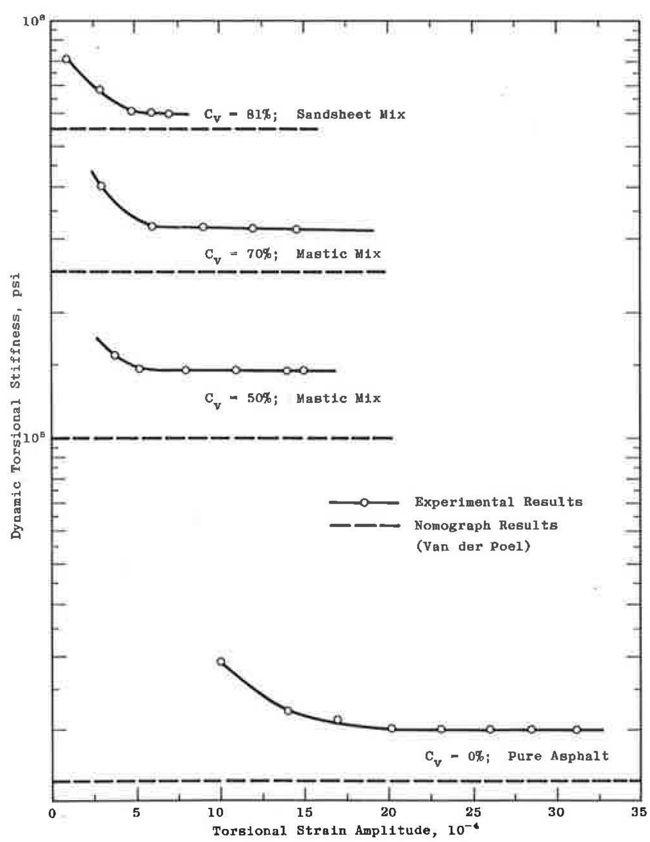


Figure 7. Variation of stiffness for different mixes at 0 C (constant frequency, 24 cyc/sec) (after Pell and McCarthy, 121).

TABLE 2

STIFFNESS DETERMINATIONS, ASPHALTIC CONCRETE TEST SPECIMENS

SAMPLE GROUP	NO. OF SPECIMENS	AVG. ASPHALT CONTENT (%)	AVG. C_v	AIR VOIDS (%)	RECOVERED ASPHALT: PEN. @ 77 F (DMM)	RECOVERED ASPHALT: R-&-B SOFT. PT. (°F)	COMPUTED S [a] (PSI × 10^5): 68 F	COMPUTED S [a] (PSI × 10^5): 40 F	MEASURED S [b] (PSI × 10^5): 68 F MEAN	MEASURED S [b]: 68 F STD. DEV.	MEASURED S [b]: 40 F MEAN	MEASURED S [b]: 40 F STD. DEV.
(*a*) SPECIMENS FROM SURFACE COURSE [c]												
1	19	5.9 [d]	0.87	8.0	46	128	3.3	15.1	1.79	0.42	6.80	1.53
2	20	5.9 [d]	0.87	8.9	36	138	5.0	14.7	1.65	0.39	7.03	1.91
3	20	5.9 [d]	0.87	7.6	52	133	3.2	11.8	1.52	0.41	7.12	1.41
4	19	5.9 [d]	0.87	8.1	59	126	3.1	12.7	1.34	0.37	5.90	1.11
Lab. Compacted	26	6.0	0.86	5.0	57	128	2.8	10.5	1.29	0.22	5.76	0.74
(*b*) SPECIMENS FROM BASE COURSE [e]												
1	12	4.6 [d]	0.87	8.8	36	130	7.4	23.2	1.57	0.42	5.95	1.54
2	12	3.6 [d]	0.88	8.0	35	130	8.9	27.4	1.39	0.26	5.66	3.14
3	8	4.6 [d]	0.88	9.4	33	140	5.8	21.0	1.47	0.41	4.40	0.90
4	10	4.6 [d]	0.88	9.1	48	129	5.8	20.1	1.42	0.42	4.96	1.22
Lab. Compacted	29	4.7	0.89	5.7	45	132	7.6	27.8	1.19	0.19	5.31	1.50
(*c*) LABORATORY SPECIMENS [f]												
Granite + 85-100 pen. asph.	76	6.0	0.85	4.5	40	121	2.8 [g]	—	2.62 [g]	0.32 [g]	—	—
Granite + 85-100 pen. asph.	14	6.0	0.85	3.6	52	128	2.7	11.5	2.06	0.58	12.6	1.82
Granite + 85-100 pen. asph.	12	6.0	0.85	3.1	52	123	3.6	—	2.10	0.42	—	—
Granite + 15 pen. asph.	12	6.0	0.85	3.6	15	161	7.8	—	5.60	0.92	—	—

[a] Stiffness determined at a 0.1-sec time of loading (from Figs. 5 and 6).
[b] Flexural stiffness measured after 200 repetitions of bending stress at 0.1-sec time in flexural fatigue apparatus development by Deacon (*25*).
[c] Surface course mixture contains an aggregate conforming to the 1960 California Division of Highways specifications for a 3/4-in. maximum size medium grading.
[d] Average asphalt content based on analysis of mixture samples obtained at central plant at time of construction.
[e] Base course mixture contains an aggregate conforming to the 1960 California Division of Highways specifications for a 2-in. maximum size material.
[f] Laboratory-prepared specimens with a gradation conforming to the 1960 California Division of Highways specification for 3/8-in. maximum size, from an investigation by Deacon (*25*).
[g] Stiffness at 75 F.

By measuring both the absolute value of the complex modulus and the phase angle over a range in frequencies, viscoelastic response of the asphaltic concrete as a function of time is determined. Through numerical procedures (*138*), it is possible to relate the complex modulus as a function of frequency to the relaxation modulus as a function of time.

Van der Poel (*156*), rather than using the mathematical transforms necessary to develop the relaxation modulus from the complex modulus, has suggested that the modulus determined from sinusoidal loading tests could be plotted as a function of time simply by taking the modulus determined at a particular frequency, ω, and plotting it on the time scale at $1/\omega$, which is a particular time (e.g., $\frac{1}{\text{radian/sec}} = \text{sec}$). The stiffness or complex modulus can then be compared with the relaxation modulus as a function of time, as shown schematically in Figure 9, in which it will be noted that $E^*(\omega)$ and $E_r(t)$ are equal only at very short and long times and differ at intermediate times. From a practical standpoint, however, van der Poel has argued that this difference is small compared to the changes which can occur in the modulus over a wide range of times and has, therefore, contended that one can determine the modulus which he has termed "stiffness" from either static (e.g., creep or relaxation) or dynamic (sinusoidal loading) tests from the quotient σ/ϵ at a particular time.

The last method to be considered is the *repeated-load* test using a pulse-type loading as shown in Figure 8e. This type of test has been utilized by Seed and his co-workers to study the resilient properties of soils (*140*). Various forms of this test can be used to measure the stiffness characteristics of asphaltic concrete at different times of loading. For example, the stiffness data presented in Table 2 were obtained by measuring the deflection of beams under pulse-type loadings of the form shown in Figure 8e and determining the flexural stiffness from a relation between load and deformation, such as

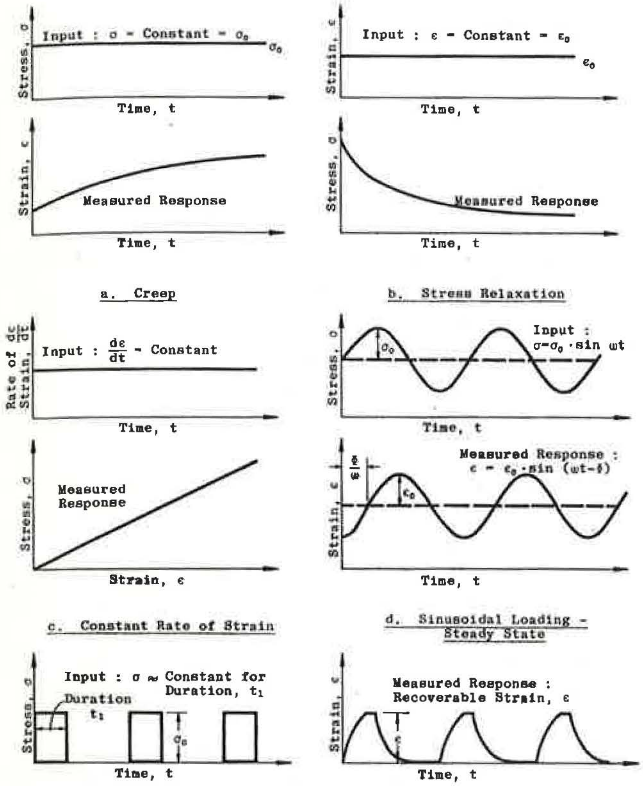


Figure 8. Types of loading to measure stiffness characteristics of asphaltic mixtures.

$$S = K \frac{P}{I\Delta} \tag{7}$$

in which

P = load applied to beam;
Δ = measured deflection (ϵ_0 in Fig. 8e);
I = moment of inertia of beam cross section; and
K = constant depending on loading geometry.

In general, it can be seen that a number of methods exist to measure the response of asphaltic concrete over a wide time scale. All of these methods have been used in recent years, and data from a number of investigations are presented in the ensuing paragraphs to establish the probable variation in stiffness characteristics and, where appropriate, the influence of mix variables on these characteristics.

As was noted earlier, the response of asphaltic concrete is not only dependent on time, but also on temperature. Procedures have also been developed by rheologists to include the influence of temperature on stiffness. A brief discussion of a systematic procedure to include these temperature effects is presented so that the data on mixtures will appear in proper perspective.

Practical limitations in the laboratory normally do not permit the complete definition of modulus or stiffness over the range of times shown schematically in Figure 9 at a particular temperature. It is, however, possible to measure

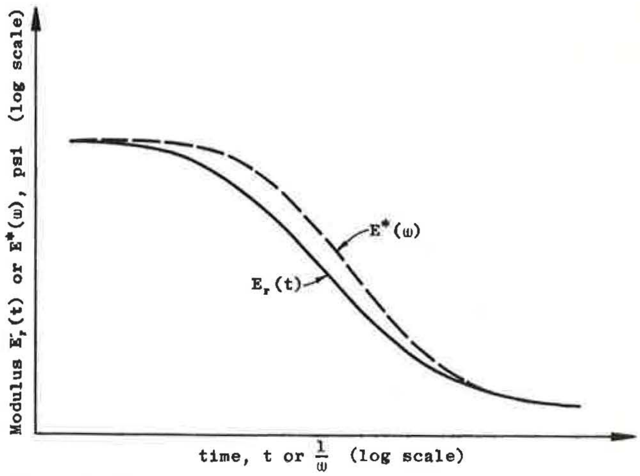


Figure 9. Comparison of the complex modulus and relaxation modulus for a linear viscoelastic material.

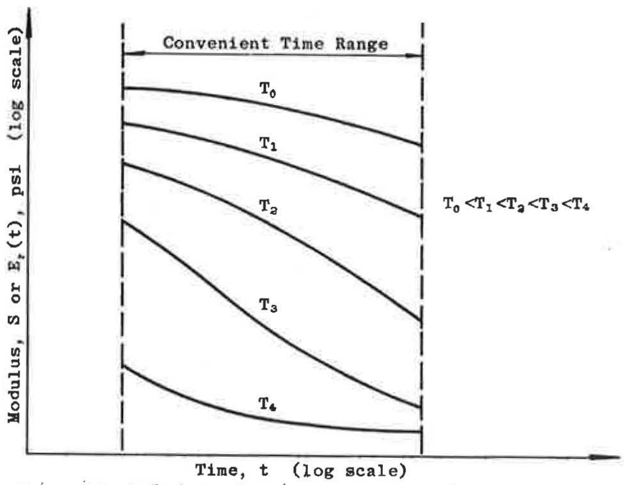


Figure 10. Influence of temperature on modulus.

the behavior for a smaller time range but at a number of different temperatures. Measured characteristics for these conditions are shown schematically by the family of curves in Figure 10.

A special case occurs if, by a temperature change, the position of the modulus curve, but not its shape, is altered on the time scale. That is, if one were to define the modulus for a wide range of times at a series of temperatures, these curves at the different temperatures are shifted only along the time axis * as shown in Figure 11. Such a material is said to exhibit "thermorheologically simple behavior." For this material, a reference temperature, T_0, may be chosen

* Theoretically, these curves should also be adjusted vertically by the ratio $T\gamma_0/T\gamma$ (where γ_0 and γ are densities at the reference temperature, T_0, and any temperature, T, respectively). This change, from an engineering standpoint, is small and is therefore not considered in this discussion. As will be seen subsequently, some investigators have made use of the correction whereas others have not.

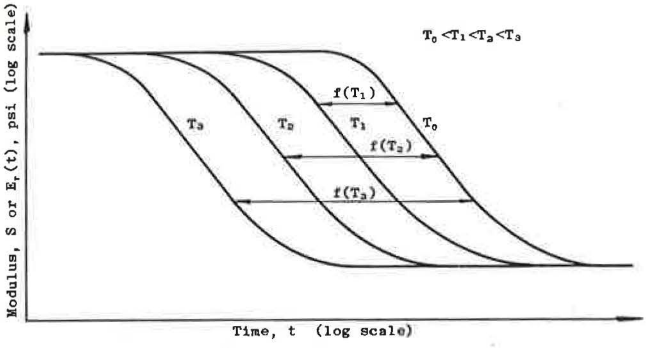


Figure 11. Effect of time and temperature on the stiffness modulus for a thermorheologically simple material (after Monismith et al., 103).

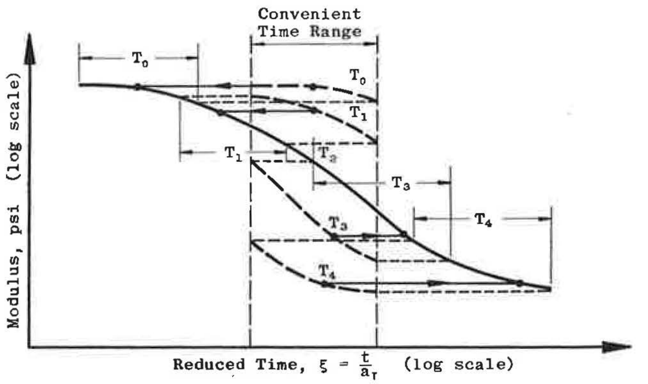


Figure 12. Shifting procedure to produce extended modulus curve at temperature T_2 *from data obtained at different temperatures.*

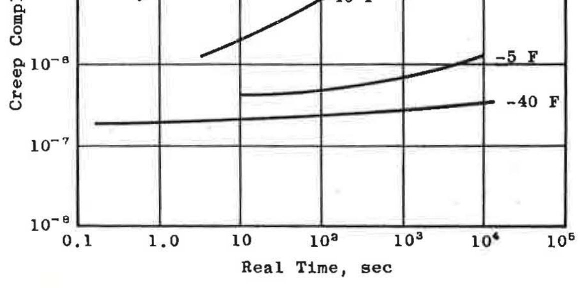


Figure 13. Creep compliance vs time at five temperatures for asphaltic concrete (after Monismith et al., 103).

and a "reduced time," ξ, defined so that the S, $E_r(t)$, or $D(t)$ curves for the various temperatures superimpose when plotted versus ξ. Thus,

$$\xi = t \exp[f(T)] \tag{8}$$

in which

ξ = reduced time corresponding to a real time, t, at a constant temperature, T; and
$f(T)$ = function of temperature giving the shift of modulus along the log t axis for temperature $T > T_0$; $f(T_0) = 0$ and $f(T_3) > f(T_2) > f(T_1)$ for $T_3 > T_2 > T_1$.

This function of temperature, $f(T)$, has been termed the shift factor and is often represented by the symbol a_T; for a particular reference temperature, the reduced time scale is simply a plot of t/a_T.

Physically, the shift factor may be defined as:

$$a_T = t_T/t_0 \tag{9}$$

in which t_T is the time required to observe a phenomenon at temperature T, and t_0 is the time required to observe the same phenomenon at some reference temperature, T_0.

Using this principle, the data shown in Figure 10 can be reduced to a particular reference temperature and the modulus defined for a much wider range in times at this temperature than could be developed practically in the laboratory. This is shown schematically in Figure 12 with T_2 as the reference temperature. In general, with data such as those in Figure 10 and the shift factor, a_T, as a function of temperature, the stiffness of an asphaltic mixture can be defined in the range of interest to road engineers.

For pure asphalts, this shift factor appears to be related to the viscosity of the asphalt (*16*). Thus, through a few tests on the material at one temperature and viscosity measurements at another, the rheologic behavior of asphalts can be defined for a wide range in times of loading and temperatures. Although the temperature dependence of mixture behavior is related to that of the asphalt, the evidence that changes in stiffness of the mixture are directly related to changes in asphalt viscosity is somewhat contradictory, as will be seen subsequently. Thus, the graphical procedure shown in Figure 12 is normally used for mixtures to develop performance at a specific temperature in terms of test data at other test temperatures.

RHEOLOGIC PROPERTIES OF ASPHALTIC CONCRETE

Although a number of researchers, in addition to van der Poel, have been concerned with establishing the rheologic characteristics of paving mixtures (e.g., Monismith and Secor (*98*) gave a summary of these investigations), this evaluation includes only the more recent studies and, in particular, those which make use of procedures previously discussed for stiffness or modulus determinations.

Secor and Monismith (*138,139*) and Monismith et al. (*103*), from creep and relaxation tests in tension, compression, and flexure, have shown that a satisfactory mea-

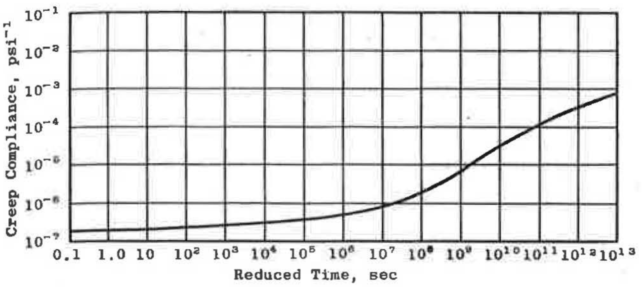


Figure 14. Master creep compliance curve at a reference temperature of —40 F.

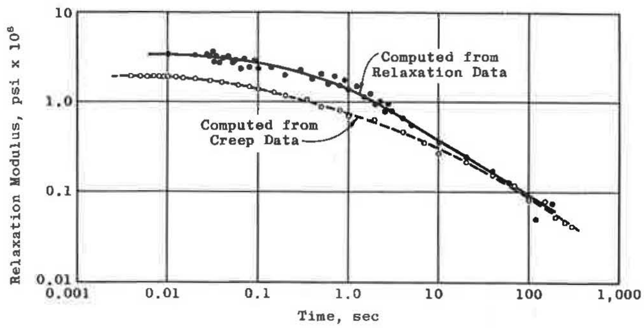


Figure 16. Comparison of relaxation modulus-time relationships as obtained from two sources (after Secor and Monismith, 138).

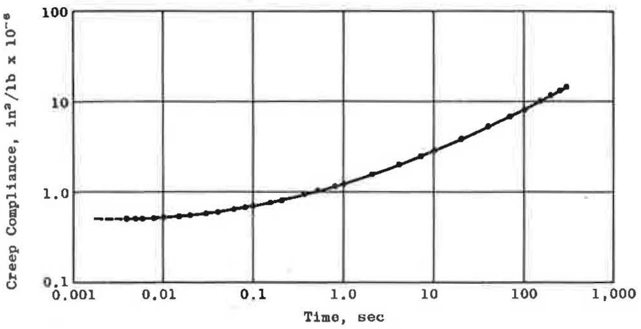


Figure 15. Creep compliance vs time from test data at 40 F (after Secor and Monismith, 138).

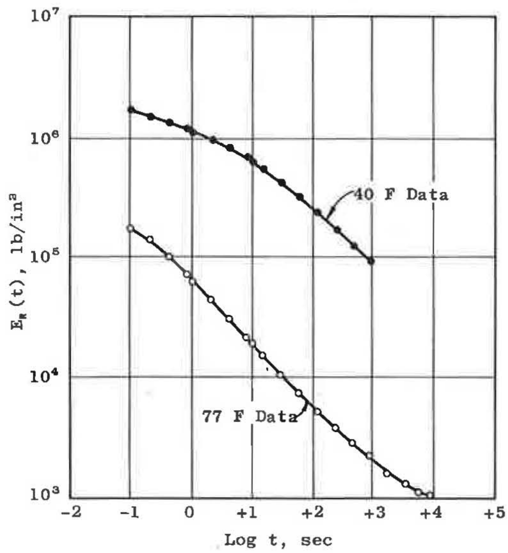


Figure 17. Reduced elastic modulus at 40 F and 77 F vs time (after Secor and Monismith, 138).

sure of stiffness, accurate at least to the level required by engineers, can be obtained by considering asphaltic concrete to be a linear viscoelastic material and one which is thermorheologically simple. Data illustrating this behavior are presented in Figures 13 through 19.

Figure 13 shows creep compliance data for asphaltic concrete in tension analogous to those presented schematically in Figure 10. These data were shifted horizontally to obtain the extended time curve of compliance versus reduced time shown in Figure 14. In these figures, it will also be noted that at low temperatures and/or short loading times this material approaches essentially elastic behavior with a compliance of the order of 2 to 3 $\times$ 10^{-7} sq in. per pound and a corresponding modulus of the order of 4 $\times$ 10^6 psi.

Figure 15 shows creep compliance data obtained from compression tests for the same mixture. These data have been converted by a numerical procedure (*138*) to develop the relaxation modulus and are compared with the relaxation modulus determined directly from stress relaxation tests in Figure 16; the computations are based on the assumption of linear viscoelastic behavior. Although there exist some discrepancies at short loading times, the comparison appears to be reasonable.

Figure 17 contains flexural stiffness data obtained in bending creep tests performed on beams of the same mixture. The modulus was computed by averaging the tensile and compressive strains (Fig. 18) at particular times after loading from

$$E_r = \frac{1}{I}\,\frac{M}{l/R} \tag{10}$$

in which

I = moment of inertia of beam;
M = applied moment;
l/R = radius of curvature $= \dfrac{\epsilon_t + \epsilon_c}{h}$;
ϵ_t = tensile strain;
ϵ_c = compressive strain; and
h = depth of beam.

The 77 F data shown in Figure 18 have been shifted to the 40 F curve to obtain an extended curve of flexural stiffness time at 40 F, as shown in Figure 19. For purposes of comparison, the moduli as a function of time

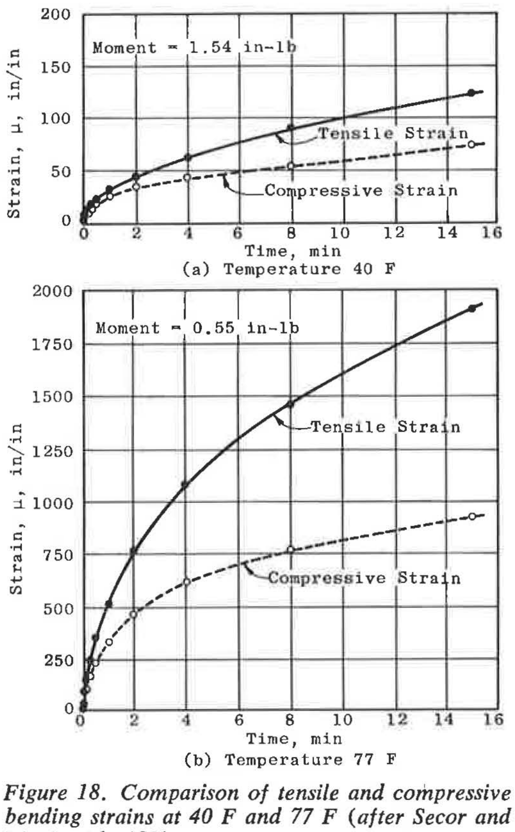


Figure 18. Comparison of tensile and compressive bending strains at 40 F and 77 F (after Secor and Monismith, 139).

computed from the creep and relaxation tests in compression (Fig. 16) have also been plotted in Figure 19 because the mixture was the same. A reasonable comparison is indicated, particularly at times less than 100 sec. This appears to lend support to the assumptions that the material is, to a first approximation, linear viscoelastic and thermorheologically simple.

Deacon (*25*) has used the load-deflection relationship in bending (Eq. 7) to define the flexural stiffness at short loading times (0.1 sec) for a mix similar to that previously reported. For a time of loading of 0.1 sec he presented the data shown in Figure 20, illustrating that the stiffness modulus at a particular time of loading is dependent on the stress level at which it is measured. Thus, one must insure that the stresses used to define the viscoelastic response are not too great, so that nonlinear effects, such as those shown in Figure 19, will be minimized.

Studies of the applicability of linear viscoelasticity and thermorheology to asphaltic mixture behavior have been conducted by Papazian (*113*) and Pagen (*111*) using sinusoidal loading and creep tests. Figure 21 shows typical data obtained in their investigations for the relationships between frequency and both the complex modulus and phase shift for an asphaltic paving mixture. It will be noted that there is a large range in the value of modulus, in this case E^*, as a function of frequency (or time).

The applicability of the time-temperature superposition principle to asphaltic mixture behavior has also been demonstrated by Pagen (*111*). Creep moduli * as a function of time of loading and temperature are shown in Figure 22. These curves have been shifted according to the graphical procedure discussed earlier to produce the composite curve at 77 F (298 K) for an extended time range shown in Figure 23. The shift factors he developed from this graphical process are shown as a function of temperature in Figure 24.

It should be noted that, although these results illustrate the behavior in axial compression (as measured by axial response to loading), these investigators have considered the possibility of three-dimensional behavior by measuring the response in a transverse direction at the same time and have defined a modulus termed the "transverse" modulus, T_c, which is the axial stress divided by the transverse strain. The form of the curve of transverse modulus as a function of time or frequency is similar to those shown in Figures 21 and 23. That this transverse modulus has the same temperature dependence as the modulus determined from axial loading is also shown in Figure 24, where the shift factor necessary to shift the transverse modulus curves to the same reference temperature, 77 F, has also been plotted as a function of temperature. As will be seen subsequently, the transverse modulus in conjunction with the modulus determined from axial response can be used to define Poisson's ratio, which is, in turn, necessary to determine the response of the material to a three-dimensional state of stress.

Although the temperature dependence of mixtures is related to the temperature susceptibility of the asphalt comprising the mixtures, current methods of asphalt viscosity measurement may not allow a direct determination. This fact is illustrated by some data presented by Pagen (*110*) and shown in Figure 25.

In many of the reported data on the viscoelastic response of paving mixtures, the behavior is defined in terms of a single-load application (e.g., Fig. 13 through 19). Pagen and Ku (*109*) have demonstrated that if the response is measured after a number of load applications (mechanical conditioning), such as that shown in Figure 26, the behavior of the material tends to be more reproducible, and linear viscoelastic theory may be more appropriately used to define the behavior of the asphaltic mixture. Results of 6 cycles of creep loading illustrating this point are shown in Figure 27. From a field performance standpoint, because the highway pavement is subjected to many repetitions of loads, this mechanical conditioning should probably be a necessary part of a testing program.

---

* The creep modulus is defined as

$$E_c(t) = \frac{\epsilon(t)}{\sigma_0}$$

in which

$E_c(t)$ = creep modulus as a function of time;
$\epsilon(t)$ = strain as a function of time; and
σ_0 = constant stress.

It will be noted that this is just the inverse of the compliance defined by Eq. 3.

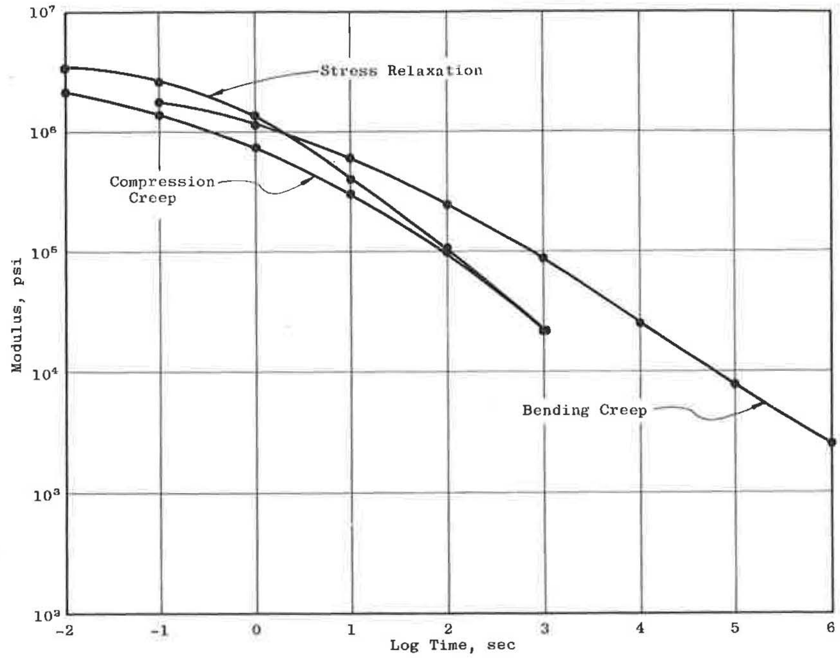


Figure 19. Comparison of moduli determined in compression and flexure at 40 F.

The influence of mix variables on creep behavior has also been reported by Pagen and Ku (*109*). In this investigation, a summary of which is included in Figure 28, the effects of asphalt type (temperature susceptibility characteristics), aggregate gradation, and aggregate type have been studied. From an examination of Figure 28, aggregate gradation appears to have more of an influence than aggregate type (all mixes at same asphalt content), particularly at longer times, and the temperature susceptibility characteristics of the asphalt may have an influence at the longer times of loading.

Krokosky et al. (*79*) have indicated that asphaltic concrete exhibits nonlinear and nonviscoelastic behavior in compression, the degree of which is dependent on the magnitude of the deformation, particularly that associated with the aggregate. In stress relaxation, the aggregate movement appears to be the least, whereas in constant rate of strain it appears to be the largest. Thus, the deviation from linear behavior is least in stress relaxation, more so in creep, and the largest in constant rate of strain. These investigators have also presented techniques for handling, at least approximately, this nonlinear behavior through formulations similar to those presented by Smith (*148*) for elastomers.

These investigators have also examined the applicability of time-temperature superposition. Data illustrating the influence of temperature on the behavior of mixes in both creep and relaxation are shown in Figure 29, in which the a_T values necessary to shift both creep and relaxation data to master curves at 40 F have been plotted. It will be noted that different results are obtained for a_T as a function of temperature, depending on the type of test. Also plotted in Figure 29 are a_T values obtained by Brodnyan (*16*) for an asphalt similar to that used in the mixes in their study. In addition, a_T values based on mixture viscosities determined from the slopes of the creep curves at longer times are also shown. These data indicate that the influence of temperature on mixture behavior is related to the asphalt but that it is not simply determined in all instances by a viscosity relation based on the viscosity of the asphalt.

Davis et al. (*24*) have indicated that the type of mineral

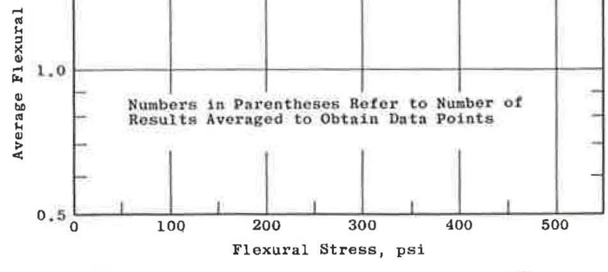


Figure 20. Relation between average flexural stiffness and flexural stress at 75 F, 0.1-sec loading time (after Deacon, 25).

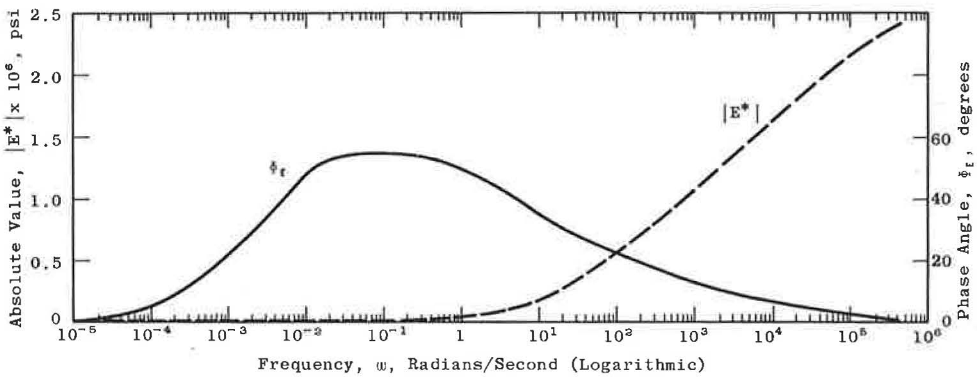


Figure 21. Magnitude and phase angle of complex elastic modulus E*, *as function of angular frequency at 77 F (adapted after Pagen,* 111).

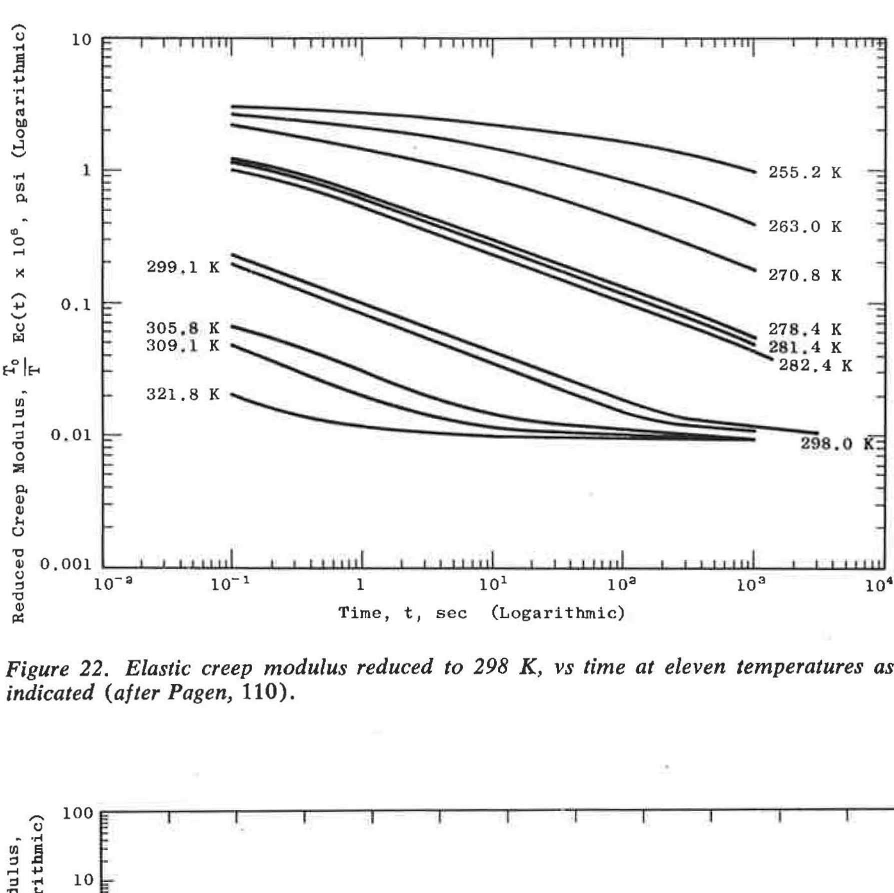


Figure 22. Elastic creep modulus reduced to 298 K, vs time at eleven temperatures as indicated (after Pagen, 110).

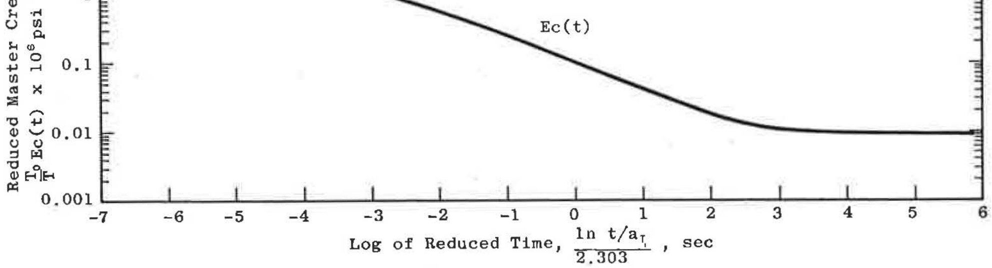


Figure 23. Composite master creep modulus obtained by time-temperature superposition principle, representing viscoelastic behavior over an extended time scale at 298 K (after Pagen 110).

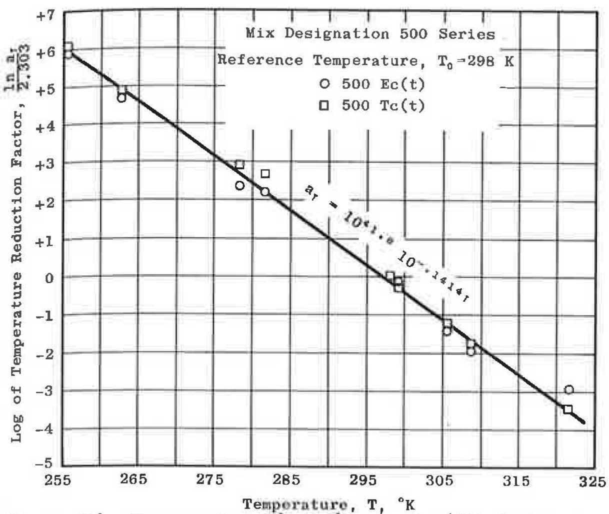


Figure 24. Temperature dependence of shift factor, a_T*, evaluated from reduced elastic and transverse creep moduli (after Pagen,* 110).

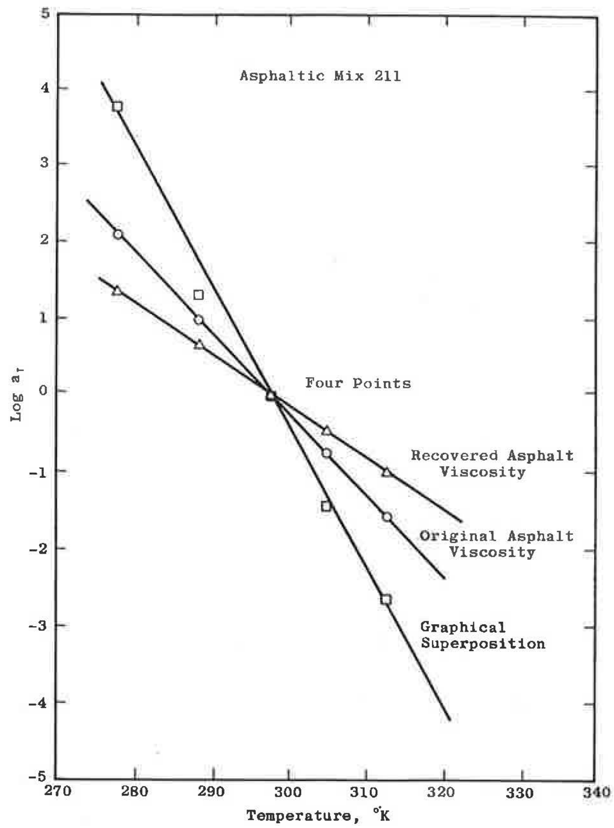


Figure 25. Typical results of log a_T *vs temperature (adapted after Pagen,* 110).

filler influences the response of asphaltic mixtures at different temperatures. This is illustrated by the data presented in Figure 30. Although the shift factors for a mix containing limestone filler appear to give the same temperature dependence as predicted by viscosity data for the asphalt, the response to temperature is influenced by the presence of asbestos.

Krokosky and Chen (*78*) have presented data indicating that the shift factor is not too dependent on asphalt content, at least within a range of asphalt contents and temperatures that would be considered for field application. This is shown in Figure 31.

In general, these investigators (*24,78,79*) indicate that time-temperature superposition is valid for asphaltic mixtures, at least to an engineering approximation.

DEFINITION OF THREE-DIMENSIONAL RESPONSE

As noted at the outset of this discussion, the asphaltic mixture in service is, in the majority of instances, subjected to a three-dimensional state of stress. Thus, to define this behavior, additional measures of response to

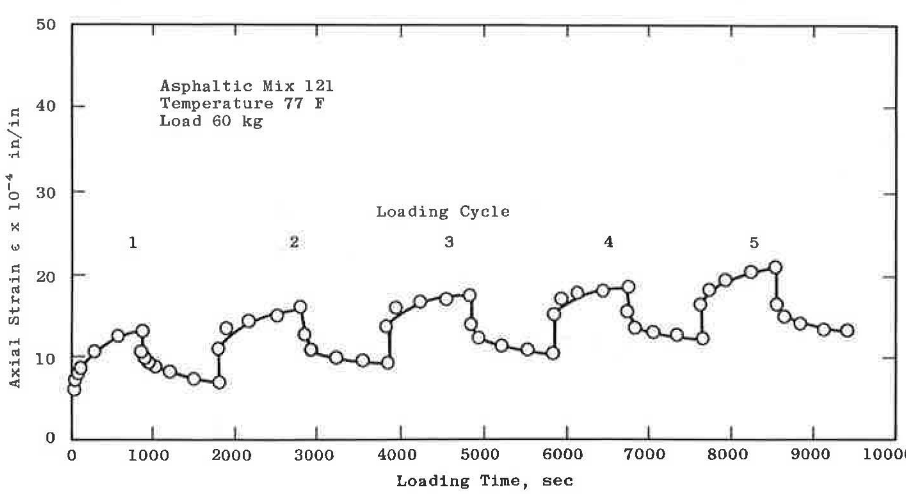


Figure 26. Creep strain under repeated loading (60-kg load) (after Pagen and Ku, 109).

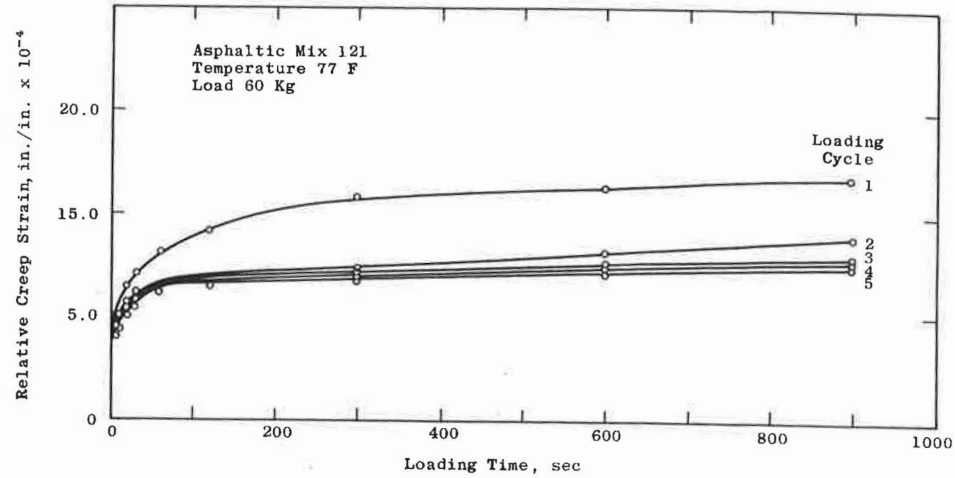


Figure 27. Relative creep strain vs loading time (60-kg load) (after Pagen and Ku, 109).

stress and strain beyond that developed for uniaxial loading must be considered.

For an isotropic viscoelastic material, it can be shown that:

$$G^* = \frac{E^*}{2(1 + \nu^*)} \tag{11}$$

in which G^* is the complex shear modulus and ν^* is the complex Poisson's ratio.

In addition,

$$K^* = \frac{E^*}{3(1 - 2\nu^*)} \tag{12}$$

in which K^* is the complex bulk modulus.[a] By combining Eqs. 11 and 12, a relation between E^*, ν^*, G^*, and K^* can be determined; that is,

[a] Bulk modulus $= \frac{\text{hydrostatic pressure}}{\text{change in vol. per unit vol.}}$

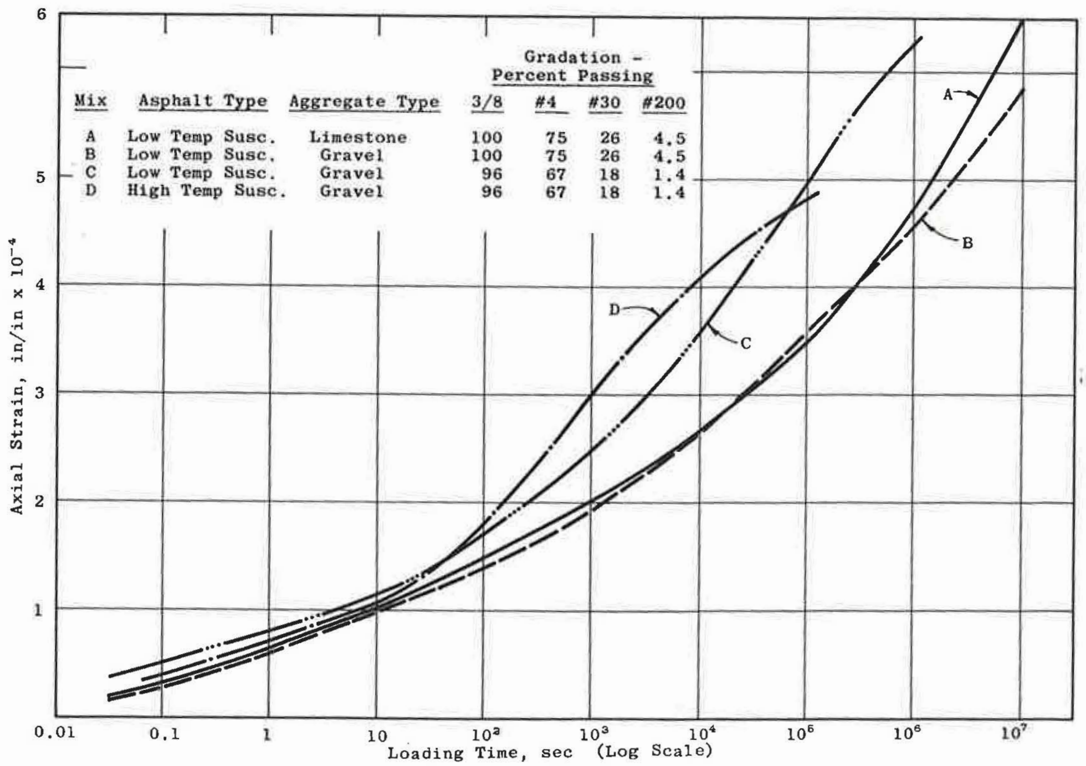


Figure 28. Influence of mix variables on creep behavior of asphaltic concrete at 77 F (after Pagen and Ku, 109).

$$\frac{1}{E^*} = \frac{1}{3G^*} + \frac{1}{9K^*} \tag{13}$$

Thus, to define three-dimensional response, two of the previously noted quantities must be ascertained; e.g., E^* and ν^*. Similar relationships can be developed in terms of time, but their form is not quite so simple.

Papazian (*113*) has demonstrated that the determination of a time-dependent Poisson's ratio would permit the extension of data obtained for a uniaxial state of stress to predict three-dimensional behavior. He has suggested that measurements of the transverse strain, as well as the axial strain, in a constant axial strain (creep) test would permit this determination.

Both Papazian (*113*) and Pagen (*111*) have indicated that the complex Poisson's ratio can be determined at a particular frequency from measurements of the transverse modulus as well as the modulus in the direction of stress application; that is,

$$\nu^* = \frac{T^*}{E^*} \tag{14}$$

in which T^* is the complex transverse modulus. As noted earlier, Figure 24 shows that the temperature dependence of the transverse modulus is the same as that of the complex modulus E^* for the mixes studied by Pagen (110).

Monismith and Secor (*98*) have presented data indicating that Poisson's ratio is temperature dependent, increasing as the temperature increases. Data determined at small deformations in constant rate-of-strain triaxial compression tests are presented in Table 3. To a first approximation, it would appear that Poisson's ratio is essentially independent of load rate at a particular temperature and at temperatures above 75 F approaches a value of 0.5. For this situation, Eq. 11 reduces to

$$E^* \approx 3G^* \tag{11a}$$

which, in turn, simplifies the treatment of three-dimensional problems and permits the use of data determined from uniaxial compression and tension tests or flexural tests such as those previously discussed to be used directly in the solution of the problem.

It should again be emphasized that these results are applicable to small deformations. When the deformations become large, it is possible that dilation (volume increase)

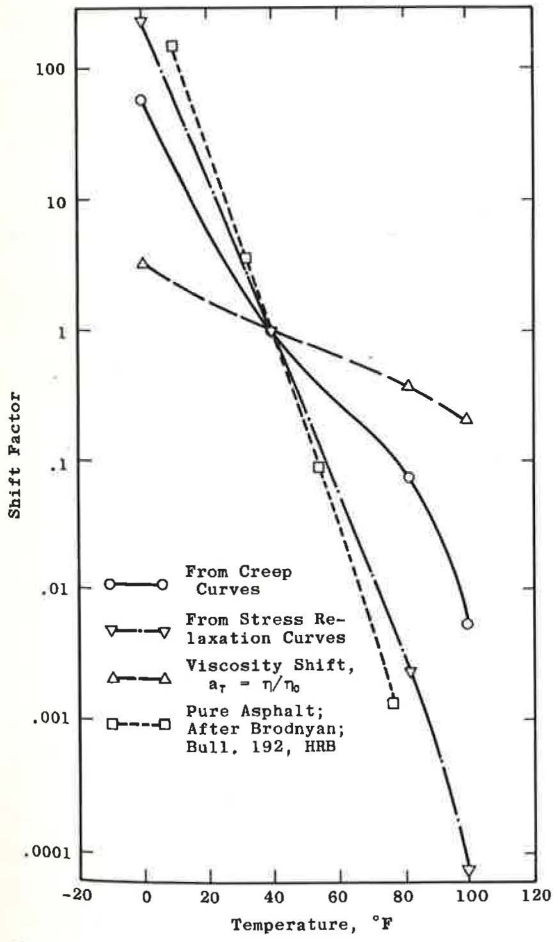


Figure 29. Creep test; shift values as function of temperature (after Krokosky, Tons, and Andrews, 79).

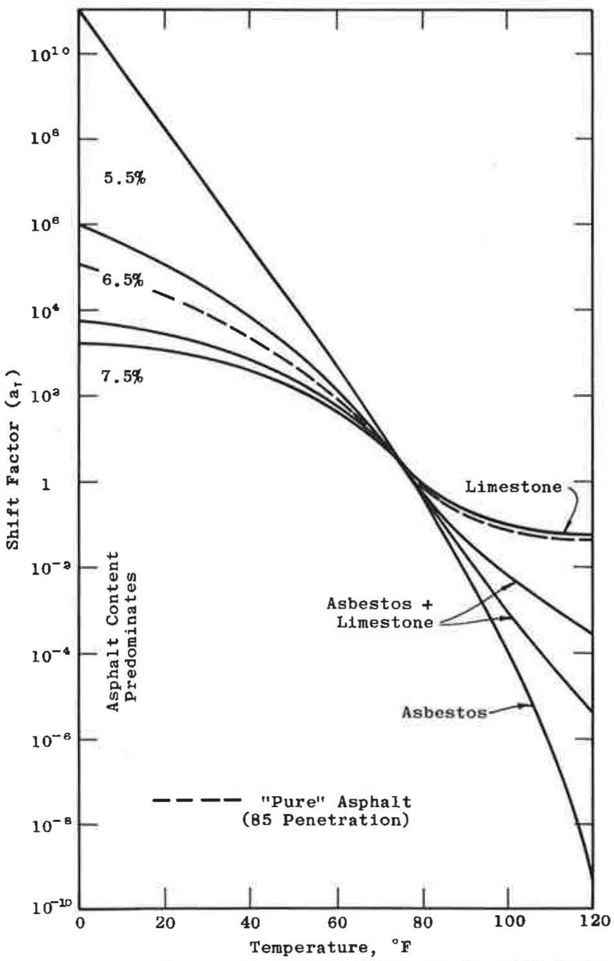


Figure 30. Influence of mineral fillers on the shift factor for asphaltic concrete mixtures in tension (after Davis, Krokosky and Tons, 24).

TABLE 3

POISSON'S RATIOS COMPUTED FROM VOLUME CHANGE MEASUREMENT (CONSTANT-RATE-OF-STRAIN TESTS)[a]

TEMP. (°F)	LOAD RATE (IN./MIN)	POISSON'S RATIO
40	0.01	0.371
40	0.10	0.358
40	1.00	0.305[b,c]
77	0.01	0.492
77	0.10	0.484
77	1.00	—[b]
140	0.01	0.495
140	0.10	0.498
140	1.00	—[b]

[a] After Monismith and Secor (*98*).
[b] Volume change difficult to record at this load rate.
[c] Approximate.

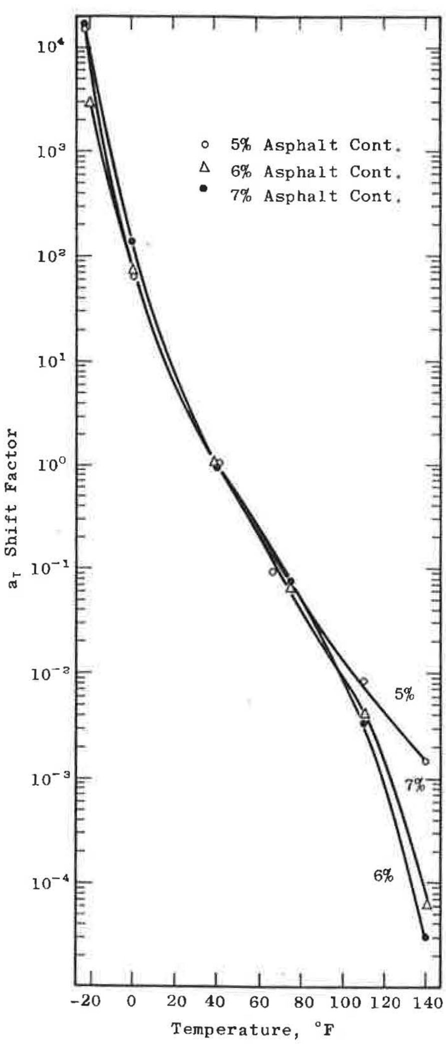


Figure 31. Shift factor a_T, *for compression stress relaxation (after Krokosky and Chen* (78) *by permission).*

will occur, and Poisson's ratio will exceed 0.5. There is some evidence of this type of behavior in data presented by Secor and Monismith (*98*).

SUMMARY

In this chapter, a summary of investigations on the rheologic characteristics of asphaltic mixtures has been presented. Table 4 summarizes the various methods used to measure these characteristics. In general, it has been shown that the stiffness of asphaltic concrete is both time-of-loading and temperature dependent and can vary by a factor of about 10^3 over the range of times and temperatures that will be expected in its application in service; that is, from about 4×10^6 psi to 10^3 psi.

Van der Poel and Heukelom and Klomp have suggested a relatively simple procedure to define stiffness of mixtures in terms of the stiffness of the recovered asphalt and volume concentration of the aggregate. This method is limited, however, to well-compacted mixtures (approximately 3 percent air voids) with dense-graded aggregates.

A number of investigators have shown that the response of asphaltic concrete can be approximated over a wide time scale by the application of the principles of linear viscoelasticity and that the effects of temperature on this response can be determined through use of the time-temperature superposition principle. Krokosky et al. have demonstrated, however, if the deformations are sufficiently large in compression, the behavior of the material is nonlinear and even nonviscoelastic, although the time-temperature superposition principle still appears to be applicable, at least to an engineering approximation.

Although no definitive guides have thus far been presented in the literature as to the limit of applicability of these principles, recently Alexander (*3*) has given some indication that, so long as the deformations do not exceed about 0.1 percent, linear viscoelasticity can be satisfactorily used. Actual field measurements (Table 21) and theoretical computations (Figs. 108 and 109) indicate 0.1 percent is realistic in terms of the pavement system.

Secor and Monismith have indicated that a difference in behavior can be expected in tension and compression at longer times of loading and higher temperatures. It would appear, however, that this difference can be minimized by measurements of flexural response rather than through either compression or tension tests. That is, if one is concerned with the stiffness of an asphaltic paving slab under moving traffic, flexural measurements may be the most appropriate method to define the rheologic characteristics.

Because of the limitations of small deformations, it is possible that linear viscoelastic theory may not provide the engineer a method to predict the accumulation of measurable deformations leading to rutting. However, it does have the potential to assess stiffness characteristics (i.e., stress vs strain characteristics) for a wide range in times of loading and temperatures, and thus is extremely useful in evaluating the load transmission characteristics of asphaltic concrete in pavement structures.

TABLE 4

SUMMARY OF METHODS TO MEASURE RHEOLOGIC BEHAVIOR OF ASPHALTIC CONCRETE

METHOD OF TEST	INPUT	MEASURED RESPONSE	MEASURE OF RHEOLOGIC BEHAVIOR	REMARKS
Creep: (a) Axial loading (b) Bending	Tension and compression: Constant stress, σ_0 Flexure: Constant moment	Strain as a function of time, $\epsilon(t)$	(a) Creep compliance, $D(t) = \frac{\epsilon(t)}{\sigma_0}$ (b) Creep modulus, $E_c(t) = \frac{\sigma_0}{\epsilon(t)}$ (c) Creep modulus in flexure, $E_B(t) = \frac{1}{I} M \frac{[\epsilon_t + \epsilon_c](t)}{h}$	(a) Necessary to use superposition principle[a] to relaxation modulus, $E_r(t)$, from compliance, $D(t)$. (b) $E_c(t) = E_r(t)$ only at short and long loading times. (c) $E_B(t) = E_r(t)$ only at short loading times for asphaltic concrete
Relaxation: (a) Axial loading	Tension and compression: Constant strain, σ_0	Stress as a function of time, $\sigma(t)$	Relaxation modulus, $E_r(t) = \frac{\sigma(t)}{\epsilon_0}$	(a) Analogous to van der Poel's stiffness. (b) Difficult to perform true stress relaxation test on asphaltic concrete.
Constant rate-of-strain: (a) Axial loading	Tension and compression: Constant rate-of-strain, $d\epsilon/dt$	Stress and strain, σ and ϵ	Relaxation modulus, $E_r(t) = d\sigma/d\epsilon$ at different values for $d\epsilon/dt$	
Dynamic loading (a) Axial loading	Tension and compression: Sinusoidally varying stress, $\sigma = \sigma_0 \sin \omega t$; or sinusoidally varying strain $\epsilon = \epsilon_0 \sin \omega t$. Range in frequencies	For stress input: Sinusoidally varying strain, $\epsilon = \epsilon_0 \sin(\omega t - \Phi)$ at particular frequency, where Φ = phase shift ω = frequency	Complex modulus, $\lvert E^* \rvert = \sigma_0/\epsilon_0$ and phase shift Φ for a range in frequencies	(a) $\lvert E^* \rvert = E_r(t)$ only at short and long loading times. Necessary to use another form of superposition principle to determine $E_r(t)$ for intermediate times. (b) By plotting $\lvert E^* \rvert$ as a function of $1/\omega$, a curve similar in shape to $E_r(t)$ is obtained; will be displaced somewhat from $E_r(t)$ curve for intermediate times as noted in (a) above.
Repeated loading: (a) Axial loading (b) Flexure	Compression: Axial stress, σ_d Flexure: Applied load, P	Compression: Recoverable strain after a specific number of load applications, ϵ_R Flexure: Recoverable deflection after a specific number of load applications, Δ_R	Compression: Resilient modulus: $M_R = \sigma_d/\epsilon_R$ Flexural stiffness: $S = K \frac{P}{I\Delta_R}$ where K = constant depending on loading conditions	
Stiffness (according to van der Poel)	Penetration and ring-and-ball softening point of recovered asphalt; volume concentration of aggregate, C_v		Stiffness, $S(t,T) = \sigma/\epsilon$	(a) Analogous to relaxation modulus.

[a] One statement of superposition principle: $\int_0^t D(t-\tau)E(t)d\tau = \int_0^t \epsilon(t-\tau)D(t)\tau = t.$

Although there is evidence that the change in stiffness of mixtures is related to the change in stiffness of the asphalt contained in the mixture, one cannot simply state that this change can be defined through measurements of the viscosity of asphalt at different temperatures; e.g., Pagen's research. Thus, until more evidence becomes available, it appears that stiffness measurements should be made at different temperatures on the mixture and the graphical procedure for shifting data (Fig. 12) be utilized to expand the time scale at a particular temperature if required.

To measure the stiffness characteristics of asphaltic concrete in pavements subjected to many repetitions of traffic, the data of Pagen and Ku would indicate that the behavior should be defined after a series of conditioning loads have been applied. In vibratory- and repeated-load testing this is not a problem, because measurements of stress and strain are taken when the so called "steady state" has been obtained. If the response is to be based on results of creep, relaxation, or constant rate-of-strain tests, some "conditioning" (load on deformation) should be utilized prior to the definition of the response. In the case of creep measurements, for example, this "conditioning" appears to require 5 to 10 load applications.

To predict three-dimensional response, it is necessary to define at least one other parameter in addition to the stiffness measured in simple compression, tension, or flexure. Papazian and Pagen have suggested use of the transverse modulus obtained from deformation measurements in the direction normal to loading in either vibratory or creep tests. They have presented expressions indicating that a time- and temperature-dependent Poisson's ratio can be developed from the ratio of the transverse modulus to the stiffness modulus determined in the direction of loading.

Monismith and Secor have indicated that Poisson's ratio may vary from about 0.3 to 0.5 for small deformations. At colder temperatures, the ratio appears to be dependent on the rate of loading. Thus, a value of 0.3 might be appropriate at cold temperatures (less than 40 F) and times of loading of 0.1 sec. At higher temperatures and slow rates of loading, a value of 0.5 would seem reasonable.

In subsequent sections of this report, practical examples of the use of these stiffness measurements are presented. In general, when these applications are indicated, it will be noted that the stiffness corresponding to a particular time of loading and temperature will be used in an expression of structural response developed on the basis of elastic rather than viscoelastic theory. From an engineering standpoint, this is necessary because theoretical developments related to the determination of response of structures in which the components are represented by viscoelastic materials are limited in scope. The use of existing elastic solutions with time- and temperature-dependent stiffness moduli has already been suggested by a number of investigators (e.g., *22, 133, 139*).

DESIGN IMPLICATIONS

The preceding discussion of present developments relating the application of rheology and rheological properties of materials to the performance of asphaltic surfacing indicates that this factor has limited direct application. Potentially, a knowledge of these properties should enable the engineer to estimate the stress-strain relationships under given load-time-temperature conditions. Mathematical solutions have proven to be difficult to handle and not always reliable; therefore, it is suggested that, for the present, the theories of viscoelasticity should defer to those of elasticity as a factor in the design of asphaltic surfacings.

However, a knowledge of the rheological properties of asphalt and asphaltic concrete can be a most important tool in selecting appropriate values of stiffness to be used as a modulus of elasticity in the elastic theory. As shown in Figure 2, this property can be used to represent a full range of values from the elastic case, where stiffness approaches 4×10^6 psi, to viscous behavior, where stiffness is approximately 1×10^3 psi.

Careful consideration must be given to the conditions under which the stiffness value is to be selected. For instance, what temperature, time, and load characteristics will be most prevalent or dominating under field conditions? In all probability, more than one set of conditions should be considered, usually representative of seasons of the year. This poses only a minor problem insofar as the determination of the appropriate stiffness value is concerned. By using the principles of time-temperature superposition, it is possible to obtain stiffness values over a wide range of conditions. It must be remembered that stiffness is not only changing seasonally, but also daily; i.e., the daily climatological changes at a given location will cause changes in stiffness. Surfacing temperature also varies with the depth of the asphaltic concrete layer, thus introducing another factor influencing stiffness. Therefore, the difficulty comes in integrating these changes for a specific situation.

For the present, the engineer must resort to realistic simplifications in choosing time and temperature parameters that will be representative of field conditions over the long-term period of expected performance. To do this intelligently, it will be necessary to make the necessary observations and measurements in the field to establish the test conditions.

The concept of using a temperature shift factor need not be limited to stiffness. It should also be applicable to measurements of stability and cohesion if a master curve relating time and temperature is considered important.

CHAPTER THREE

FRACTURE STRENGTH

In Chapter Two it was demonstrated that the stress-strain characteristics of asphaltic concrete are both time-of-loading and temperature dependent. From the available data, it appears that the fracture or breaking strength of asphaltic concrete also depends on these parameters. As in previous sections, the characteristics of the components as well as those of the mixtures are considered in the development of information on this factor.

ASPHALT

Van der Poel (*156*) has shown that, at short times of loading and low temperatures, the breaking strength of asphalt in tensile creep tests approaches 30 kg/cm² (≈ 420 psi). This trend is shown in Figure 32, which also shows the influence of asphalt type on breaking strength, particularly as influenced by temperature. The pitch-type bitumen in this case exhibits essentially brittle behavior over a much wider range in times.

Van der Poel also presented data obtained by Lethersich (*81*) illustrating the influence of rate of loading and of viscosity on the breaking strength of asphalts. These data, reproduced in Figure 33, approach limiting values in the same range as those presented in Figure 32.

Eriksson (*34*) has also investigated the fracture strength of asphalt and has determined the tensile strength for a number of materials to be in the range 20 to 40 kg/cm² (essentially the same as that reported by van der Poel).

Rigden and Lee (*125*) have reported data for the tensile strength of both weathered and unweathered tars and asphalts to be in the range 25 kg/cm² at high rates of loading in constant-rate-of-stress tests for specimens with cross-sectional areas approaching 1 sq cm. However, they determined that the size of the specimen affects the breaking strength. This effect is shown in Figure 34. Similar size effects have been observed in other materials and have been attributed to the presence of a large number of flaws in the material in larger cross sections.

Brodnyan (*16*) has briefly presented some tension test data obtained from research sponsored by the National Asphalt Research Council on 11 representative asphalts used in the United States. These tests, like those of van der Poel and Lethersich, would be considered "in bulk" tests because the specimens were at least 3 in. long with a 0.45- by 0.25-in. cross section. Brodnyan reported values of tensile stress up to about 25 kg/cm² (≈ 350 psi) at 0 C for a gel-type asphalt.

The general observation that all bituminous binders have maximum tensile strengths of the same order of magnitude in bulk has led both van der Poel (*156*) and the British Road Research Laboratory (*126*) to the conclusion that it is possible to obtain a comparison of the susceptibility of bituminous materials to brittle fracture by measurement of their stiffness. Van der Poel suggests that brittleness effects become important when the stiffness of the material is in the range of 10,000 kg/cm² (≈ 1,400,000 psi) and greater.

Mack (*85, 86*) suggests that, because of a difference in coefficient of expansion of asphalt and aggregate, internal tensile stresses may be developed in the asphalt film which may cause rupture of the film or the formation of cavities, thereby reducing the strength of the film. To study this effect, Mack (*85*) determined the force necessary to separate in tension two steel plates cemented together by a thin film of asphalt.

In Figure 35, it will be noted that an optimum film thickness in terms of tensile strength was obtained in the range of 20 to 50 microns; and for the asphalts investigated, a maximum tensile strength of the order of 14 kg/cm² (200 psi) was developed. (Tables 5 and 6 contain a summary of the standard test properties of the asphalt and values for their corresponding tensile strengths.) It should be noted

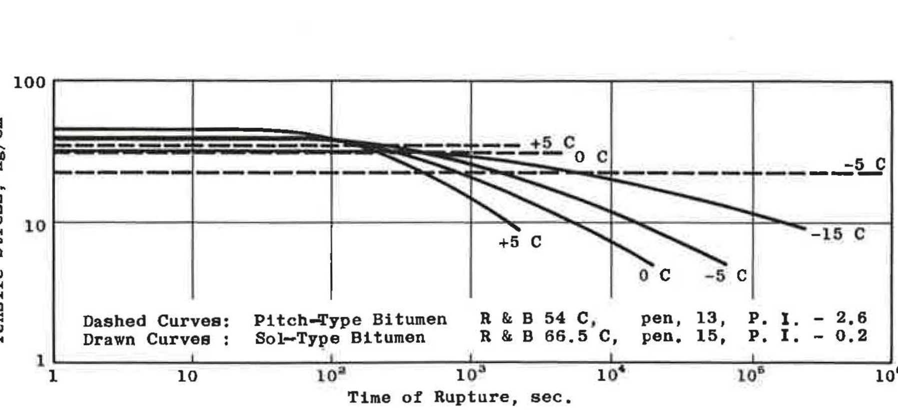


Figure 32. Breaking strength under constant tensile stress as a function of time (after van der Poel, 156).

Although there is evidence that the change in stiffness of mixtures is related to the change in stiffness of the asphalt contained in the mixture, one cannot simply state that this change can be defined through measurements of the viscosity of asphalt at different temperatures; e.g., Pagen's research. Thus, until more evidence becomes available, it appears that stiffness measurements should be made at different temperatures on the mixture and the graphical procedure for shifting data (Fig. 12) be utilized to expand the time scale at a particular temperature if required.

To measure the stiffness characteristics of asphaltic concrete in pavements subjected to many repetitions of traffic, the data of Pagen and Ku would indicate that the behavior should be defined after a series of conditioning loads have been applied. In vibratory- and repeated-load testing this is not a problem, because measurements of stress and strain are taken when the so called "steady state" has been obtained. If the response is to be based on results of creep, relaxation, or constant rate-of-strain tests, some "conditioning" (load on deformation) should be utilized prior to the definition of the response. In the case of creep measurements, for example, this "conditioning" appears to require 5 to 10 load applications.

To predict three-dimensional response, it is necessary to define at least one other parameter in addition to the stiffness measured in simple compression, tension, or flexure. Papazian and Pagen have suggested use of the transverse modulus obtained from deformation measurements in the direction normal to loading in either vibratory or creep tests. They have presented expressions indicating that a time- and temperature-dependent Poisson's ratio can be developed from the ratio of the transverse modulus to the stiffness modulus determined in the direction of loading.

Monismith and Secor have indicated that Poisson's ratio may vary from about 0.3 to 0.5 for small deformations. At colder temperatures, the ratio appears to be dependent on the rate of loading. Thus, a value of 0.3 might be appropriate at cold temperatures (less than 40 F) and times of loading of 0.1 sec. At higher temperatures and slow rates of loading, a value of 0.5 would seem reasonable.

In subsequent sections of this report, practical examples of the use of these stiffness measurements are presented. In general, when these applications are indicated, it will be noted that the stiffness corresponding to a particular time of loading and temperature will be used in an expression of structural response developed on the basis of elastic rather than viscoelastic theory. From an engineering standpoint, this is necessary because theoretical developments related to the determination of response of structures in which the components are represented by viscoelastic materials are limited in scope. The use of existing elastic solutions with time- and temperature-dependent stiffness moduli has already been suggested by a number of investigators (e.g., *22, 133, 139*).

DESIGN IMPLICATIONS

The preceding discussion of present developments relating the application of rheology and rheological properties of materials to the performance of asphaltic surfacing indicates that this factor has limited direct application. Potentially, a knowledge of these properties should enable the engineer to estimate the stress-strain relationships under given load-time-temperature conditions. Mathematical solutions have proven to be difficult to handle and not always reliable; therefore, it is suggested that, for the present, the theories of viscoelasticity should defer to those of elasticity as a factor in the design of asphaltic surfacings.

However, a knowledge of the rheological properties of asphalt and asphaltic concrete can be a most important tool in selecting appropriate values of stiffness to be used as a modulus of elasticity in the elastic theory. As shown in Figure 2, this property can be used to represent a full range of values from the elastic case, where stiffness approaches 4×10^6 psi, to viscous behavior, where stiffness is approximately 1×10^3 psi.

Careful consideration must be given to the conditions under which the stiffness value is to be selected. For instance, what temperature, time, and load characteristics will be most prevalent or dominating under field conditions? In all probability, more than one set of conditions should be considered, usually representative of seasons of the year. This poses only a minor problem insofar as the determination of the appropriate stiffness value is concerned. By using the principles of time-temperature superposition, it is possible to obtain stiffness values over a wide range of conditions. It must be remembered that stiffness is not only changing seasonally, but also daily; i.e., the daily climatological changes at a given location will cause changes in stiffness. Surfacing temperature also varies with the depth of the asphaltic concrete layer, thus introducing another factor influencing stiffness. Therefore, the difficulty comes in integrating these changes for a specific situation.

For the present, the engineer must resort to realistic simplifications in choosing time and temperature parameters that will be representative of field conditions over the long-term period of expected performance. To do this intelligently, it will be necessary to make the necessary observations and measurements in the field to establish the test conditions.

The concept of using a temperature shift factor need not be limited to stiffness. It should also be applicable to measurements of stability and cohesion if a master curve relating time and temperature is considered important.

CHAPTER THREE

FRACTURE STRENGTH

In Chapter Two it was demonstrated that the stress-strain characteristics of asphaltic concrete are both time-of-loading and temperature dependent. From the available data, it appears that the fracture or breaking strength of asphaltic concrete also depends on these parameters. As in previous sections, the characteristics of the components as well as those of the mixtures are considered in the development of information on this factor.

ASPHALT

Van der Poel (*156*) has shown that, at short times of loading and low temperatures, the breaking strength of asphalt in tensile creep tests approaches 30 kg/cm² (≈ 420 psi). This trend is shown in Figure 32, which also shows the influence of asphalt type on breaking strength, particularly as influenced by temperature. The pitch-type bitumen in this case exhibits essentially brittle behavior over a much wider range in times.

Van der Poel also presented data obtained by Lethersich (*81*) illustrating the influence of rate of loading and of viscosity on the breaking strength of asphalts. These data, reproduced in Figure 33, approach limiting values in the same range as those presented in Figure 32.

Eriksson (*34*) has also investigated the fracture strength of asphalt and has determined the tensile strength for a number of materials to be in the range 20 to 40 kg/cm² (essentially the same as that reported by van der Poel).

Rigden and Lee (*125*) have reported data for the tensile strength of both weathered and unweathered tars and asphalts to be in the range 25 kg/cm² at high rates of loading in constant-rate-of-stress tests for specimens with cross-sectional areas approaching 1 sq cm. However, they determined that the size of the specimen affects the breaking strength. This effect is shown in Figure 34. Similar size effects have been observed in other materials and have been attributed to the presence of a large number of flaws in the material in larger cross sections.

Brodnyan (*16*) has briefly presented some tension test data obtained from research sponsored by the National Asphalt Research Council on 11 representative asphalts used in the United States. These tests, like those of van der Poel and Lethersich, would be considered "in bulk" tests because the specimens were at least 3 in. long with a 0.45- by 0.25-in. cross section. Brodnyan reported values of tensile stress up to about 25 kg/cm² (≈ 350 psi) at 0 C for a gel-type asphalt.

The general observation that all bituminous binders have maximum tensile strengths of the same order of magnitude in bulk has led both van der Poel (*156*) and the British Road Research Laboratory (*126*) to the conclusion that it is possible to obtain a comparison of the susceptibility of bituminous materials to brittle fracture by measurement of their stiffness. Van der Poel suggests that brittleness effects become important when the stiffness of the material is in the range of 10,000 kg/cm² (≈ 1,400,000 psi) and greater.

Mack (*85, 86*) suggests that, because of a difference in coefficient of expansion of asphalt and aggregate, internal tensile stresses may be developed in the asphalt film which may cause rupture of the film or the formation of cavities, thereby reducing the strength of the film. To study this effect, Mack (*85*) determined the force necessary to separate in tension two steel plates cemented together by a thin film of asphalt.

In Figure 35, it will be noted that an optimum film thickness in terms of tensile strength was obtained in the range of 20 to 50 microns; and for the asphalts investigated, a maximum tensile strength of the order of 14 kg/cm² (200 psi) was developed. (Tables 5 and 6 contain a summary of the standard test properties of the asphalt and values for their corresponding tensile strengths.) It should be noted

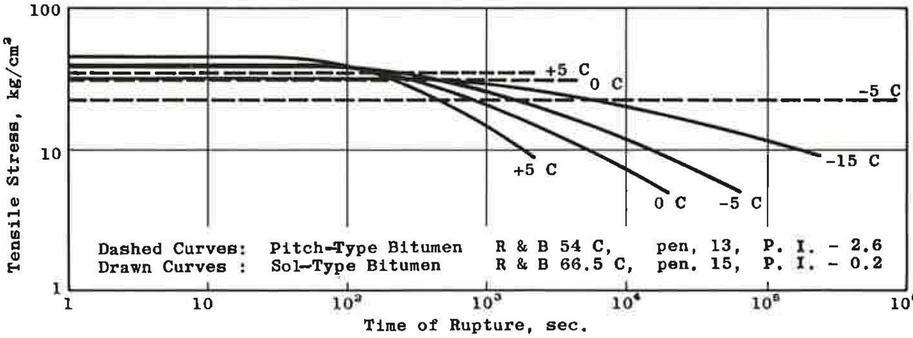


Figure 32. Breaking strength under constant tensile stress as a function of time (after van der Poel, 156*).*

TABLE 5

INSPECTION OF ASPHALT [a]

ASPHALT	SOFTENING POINT, R AND B (°F)	PENETRATION, 100 G/5 SEC 77 F	100 F	DUCTILITY (CM)	b [b]	VISCOSITY, 77 F (MEGAPOISES)	SURFACE TENSION, 77 F (DYNES/CM)
Californian reduced	110.5	87	270	100+	1.00	0.86	36.0
W. Canadian reduced	111	86.5	264	100+	1.00	0.89	34.1
W. Canadian blended [c]	111.5	86	260	100+	0.90	1.76	33.2
W. Canadian reduced + 5% GR-S	125	80	224	100+	0.90	2.50	34.1 [d]
W. Canadian oxidized	113.5	85.5	224	100+	0.76	6.62	32.0
Venezuelan reduced	115.5	85.5	223	100+	0.80	4.22	33.0
Venezuelan oxidized	118	88.5	220	100+	0.71	12.10	32.1
Coal tar pitch	116.5	50.5	205	100+	0.85	4.66	45.9

[a] Adapted after Mack (*85*).
[b] Exponent in $\tau = K(da/dt)^b$.
[c] Reduced Western Canadian crude blended with heavy lube distillate.
[d] Surface tension assumed to be same as that of Western Canadian reduced.

that these values were determined at 25 C, and the time of loading (constant rate of stress) ranged from 250 to 400 sec. Consequently, it might be argued that some flow developed, and thus the results are influenced by this factor, as were van der Poel's data at higher temperatures and longer times of loading. Mack (*86*) clearly states, however, that in these tests the material behaved as a solid rather than as a viscous fluid up to film thicknesses of 100 microns, which is beyond the range where the maximum tensile strength was obtained. Mack (*86*) has also demonstrated that, theoretically, tensile strengths of the order of 1,700 kg/cm² (≈ 24,000 psi) should be obtained independent of film thickness. Because of the residual stresses and

Bitumen	R & B	pen (25 C)	P. I. (mean)
7	60/71	15/25	0.0
8	47/53	60/80	-0.4
10	80/90	5/15	+1.4
11	82/93	20/30	+3.6
12	110/121	10/20	+5.3

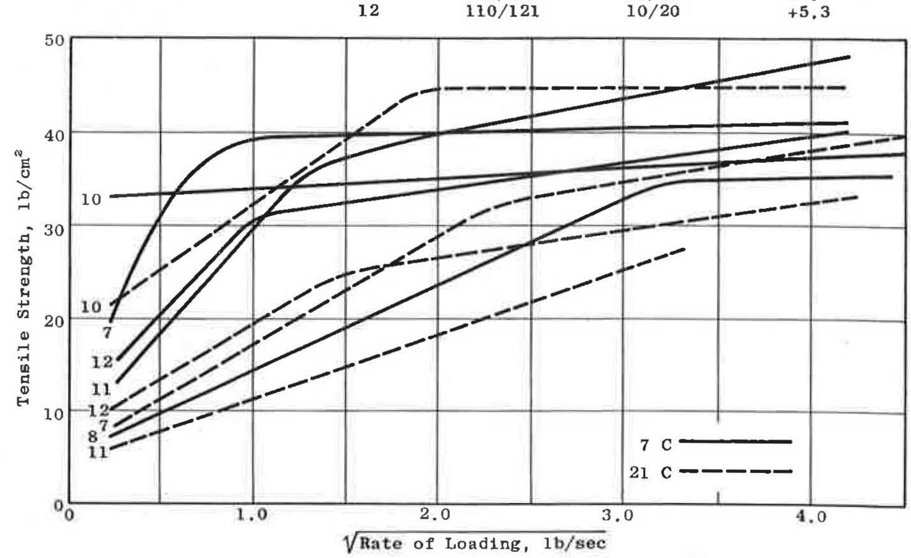


Figure 33. Breaking strength of various Venezuelan bitumens under constant rate of loading (after Lethersich, 81).

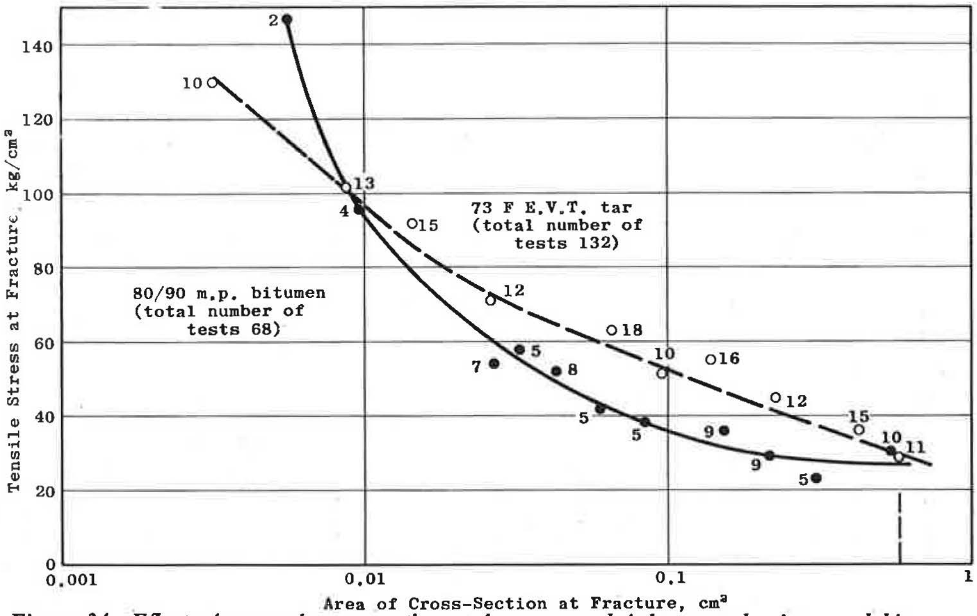


Figure 34. Effect of area of cross section at fracture on brittle strength of tar and bitumen (unrestrained conditions of test) (after Rigden and Lee, 125).

because of stress concentrations resulting from the presence of microcavities, he argues that tensile strengths cannot be expected to exceed approximately 14 kg/cm^2 ($\approx$ 200 psi)

TABLE 6

MAXIMUM FILM STRENGTH AND OPTIMUM FILM THICKNESS OF ASPHALTS[a]

ASPHALT	MAX. FILM STRENGTH (PSI)	OPT. FILM THICKNESS (CM $\times$ 10^3)	STRAIN AT RUPTURE (IN./IN.)
Coal tar pitch	276.1	3.33	7.7×10^{-4}
California reduced	211.6	2.58	6.0×10^{-4}
W. Canadian reduced	206.4	2.96	2.0×10^{-4}
W. Canadian reduced + 5% GR-S	207.0	4.00	2.3×10^{-4}
W. Canadian blended	205.1	3.38	4.9×10^{-8}
Venezuelan reduced	200.0	4.03	9.2×10^{-18}
W. Canadian oxidized	188.4	4.47	2.4×10^{-14}
Venezulan oxidized	193.5	5.14	2.3×10^{-19}

[a] Adapted after Mack (*85*).

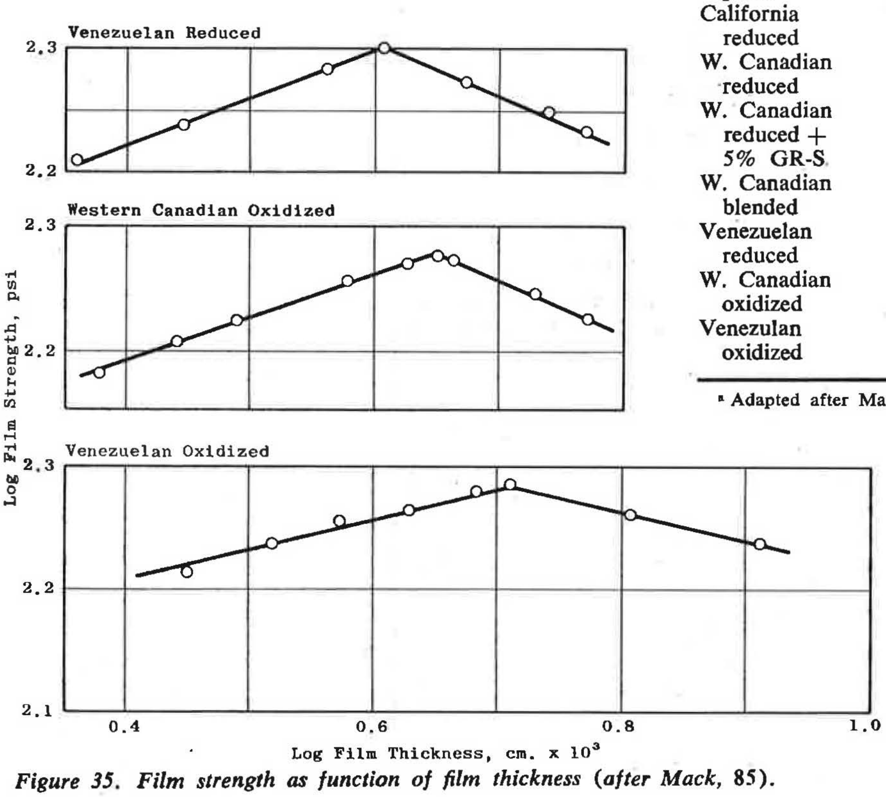


Figure 35. Film strength as function of film thickness (after Mack, 85).

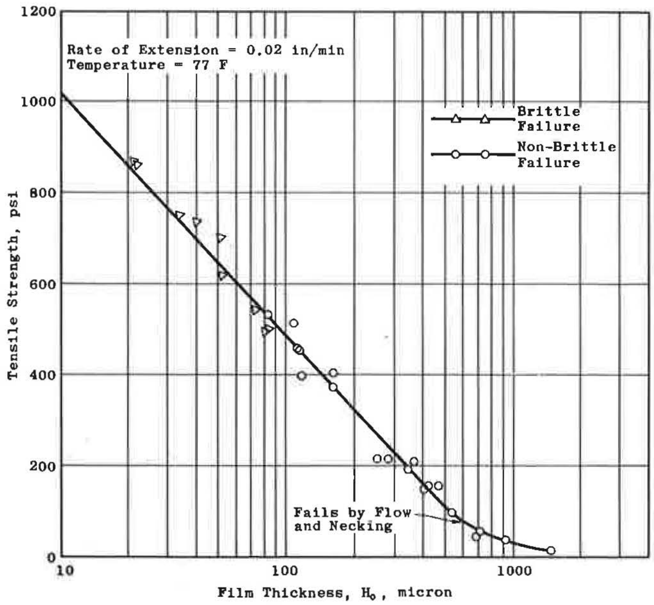


Figure 36. Influence of film thickness on tensile strength and types of failure (after Majidzadeh and Herrin, 84).

for asphalts. This value is less than those reported by van der Poel, Rigden and Lee, Eriksson, and Lethersich.

Recently, Majidzadeh and Herrin (*84*) also investigated the tensile properties of asphalts in thin films over a range in film thicknesses from 10 to 1,000 microns. Data for a 72-penetration paving asphalt are shown in Figure 36. It will be noted that values for tensile strength of the order of 900 psi (≈ 60 kg/cm²) were obtained at 77 F with film thicknesses approaching 20 microns. It is interesting to compare the approximate film thickness defining the boundary between brittle fracture and flow failure with that developed by Mack. In Figure 36, it would appear that this boundary corresponds to a film thickness of the order of 100 microns; Mack has also reported essentially the same value. It should be noted, however, that this boundary appears to be both temperature and rate-of-load dependent; this rate-of-load dependence is illustrated in Figure 37.

To attempt to place the seemingly contradictory results in proper perspective is a difficult matter. The manner in which the specimens are formed and tested appears to influence considerably the value for the tensile strength at fracture. The results of van der Poel, Eriksson, Lethersich, and Rigden and Lee are for tests on specimens essentially in bulk and should be free of conditions such as those which Mack imposed. On the other hand, the test procedures of Majidzadeh and Herrin, which appear to yield results similar to those of Mack, also may not have the same constraints as those imposed in Mack's tests. High residual stresses may have been present in Mack's specimens because they had been heated to between 125 and 150 C to develop the desirable film thickness, whereas the film thicknesses of Majidzadeh were developed by "gently heating" the blocks to allow the asphalt to flow.

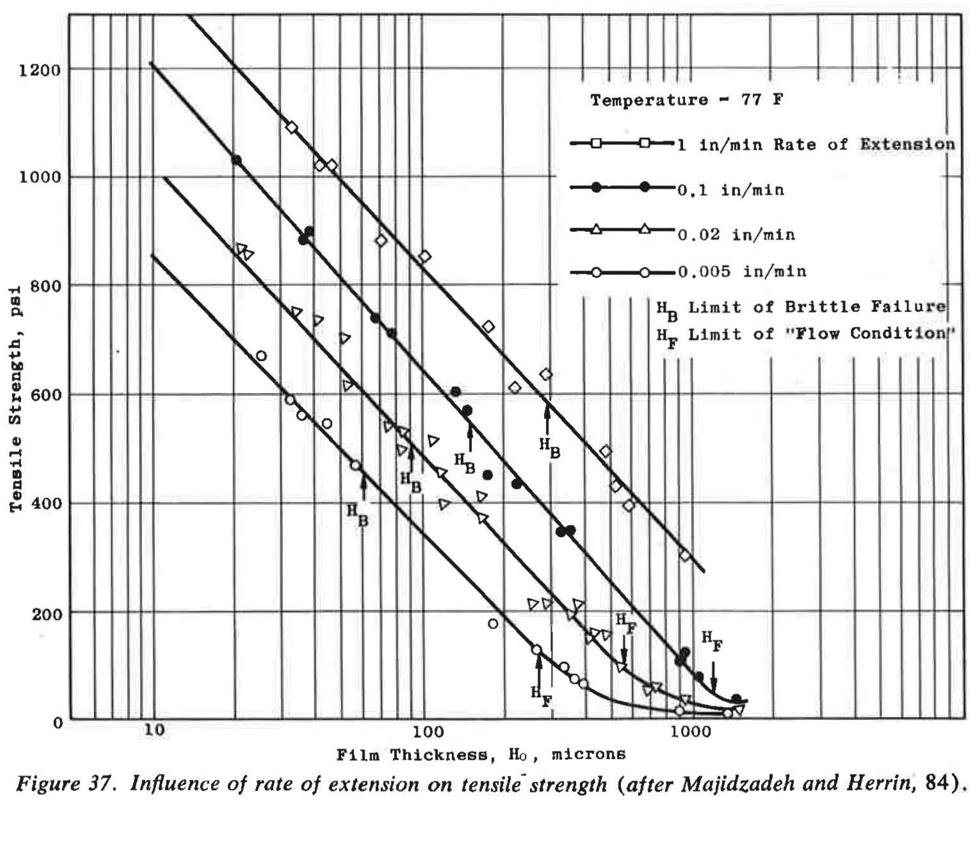


Figure 37. Influence of rate of extension on tensile strength (after Majidzadeh and Herrin, 84).

ASPHALT/MINERAL-FILLER COMBINATIONS

Rigden and Lee (*125*) have shown that the tensile strength of tars is increased by the addition of filler. The tensile strengths determined in their tests at various temperatures and filler concentrations are given in Table 7.

Data obtained by Eriksson (*34*) show similar trends for asphalt. Figure 38, for example, shows the influence of filler-bitumen ratio on the tensile strength of asphalt-filler mixtures at 1 C. It will be noted that tensile strength increases from a value slightly less than 40 kg/cm² (560 psi) to approximately 120 kg/cm² (1,700 psi). Eriksson (*34*) also notes that the sensitivity of asphalt to stress concentrations at low temperatures is decreased with the addition of filler.

TABLE 7

INCREASE IN STRENGTH OF 60° E.V.T. TAR PRODUCED BY ADDING A FINE SLATE FILLER [a]

FILLER IN MIXTURE (% BY WT.)	TENSILE STRENGTH (KG/CM²) AT 0C	AT —5C	AT —14C	FRAASS POINT (°C)
0	9	6.5	5.5	—1
10	16	—	—	—2.5
20	—	15	13.5	—1
30	21	19.5	18.5	—1.5
40	26.5	22.5	22	0
50	28.5	—	—	+2

[a] Adapted after Rigden and Lee (*125*).

ASPHALTIC CONCRETE

The tensile strength of asphaltic concrete is also both time-of-loading and temperature dependent. Van der Poel (*156*) has presented data for a sheet asphalt which illustrate that at low temperatures its strength is essentially constant, of the order of 50 kg/cm² (700 psi), and that it decreases as the temperature is increased. These data (Fig. 39) indicate both rate-of-loading and temperature dependence.

Eriksson (*34*) has presented data which indicate that the fracture strength is dependent upon mixture composition, specifically the filler-bitumen ratio (Fig. 40). At comparable filler-bitumen ratios, the tensile strength of this mixture is essentially the same as that reported by van der Poel; i.e., approximately 50 kg/cm² (700 psi). Eriksson (*34*) has also suggested that asphalts at low temperatures are sensitive to stress concentrations, inasmuch as the stress-strain relationship at these temperatures has no yield point. However, he notes that if filler is added the sensitivity of asphalt to stress concentrations at low temperatures decreases.

In another publication, Ericksson (*35*) presented data for tension tests on a particular sheet asphalt for a wide range of temperatures (Fig. 41). In general, the trend toward decreasing strength with increasing temperature is the same as that reported earlier by van der Poel. It is interesting to note, however, that for very low temperatures

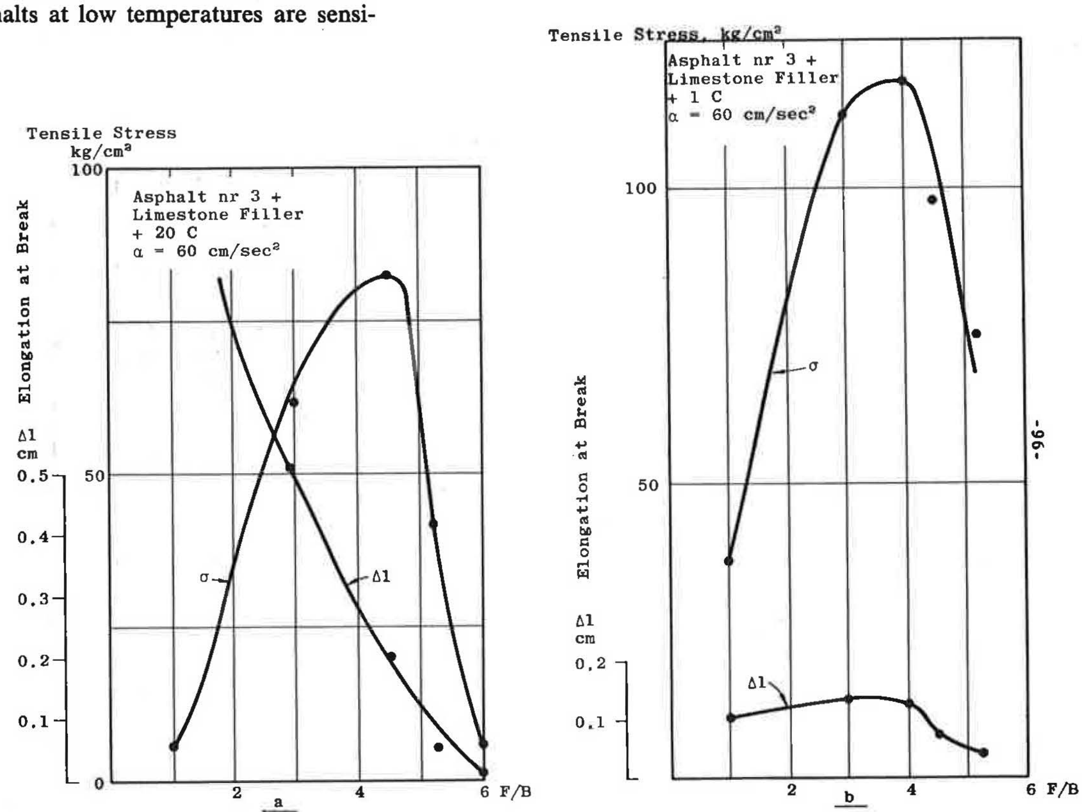


Figure 38. Variation of strength and elongation before break, with filler percentage (F/B = filler/bitumen, in parts by weight) for a mixture of filler and asphalt (nr 3); (a) +20 C, (b) +1 C (after Eriksson, 34).

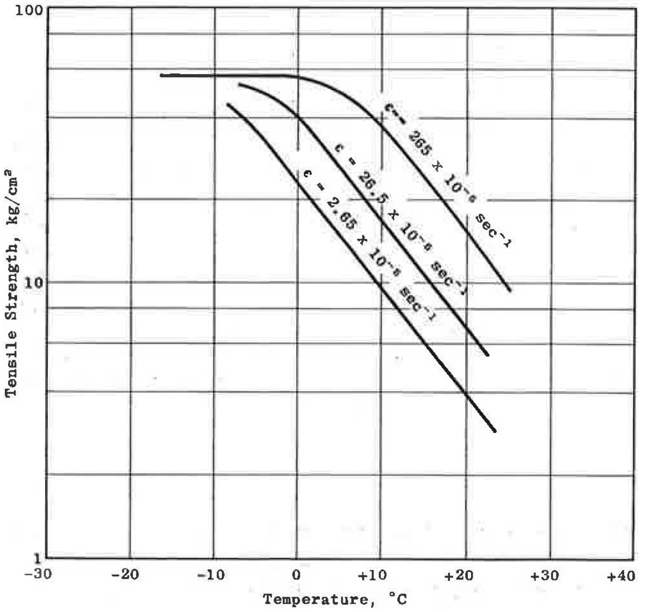


Figure 39. Tensile strength at various rates of deformation as a function of temperature (after van der Poel, 156). Mixture composition: sand (74μ —2 mm) 82.5 p.b.w., quartz filler 17.5 p.b.w., bitumen 8.5 p.b.w., sol-type pen. 60, P.I. —0.4.

the strength is also somewhat reduced. Eriksson has conjectured that this may be due to uneven distribution of stresses at these lower temperatures. Also, it will be noted that the maximum value of tensile strength is of the order of 50 kg/cm² (≈ 700 psi) for this mixture.

Tons and Krokosky (*151*) have also presented data showing the influence of mixture composition, rate of loading, and temperature on the tensile strength of asphaltic concrete. Utilizing ¼-in. maximum size aggregate, tensile strengths as high as 40 kg/cm² (≈ 600 psi) were obtained with mixtures containing a combination of limestone dust and asbestos as the mineral filler. Temperature and rate-of-loading effects similar to those reported by Eriksson were obtained. Typical data from their investigation are shown in Figure 42. Although not shown, it should also be noted that, depending on the asphalt content and the type of mineral filler, maximum values for tensile strength ranged from about 20 to 40 kg/cm² (≈ 300 to 600 psi).

Heukelom and Klomp have recently presented data (Table 8) covering a range of mixture compositions in which tensile strengths as high as 100 kg/cm² were reported. Strain at break is also given in Table 8 with a minimum value on the order of $1{,}100 \times 10^{-6}$ in. per inch.

Monismith et al. (*102*) have presented data from creep and constant-rate-of-strain tension tests on a single asphaltic concrete at one asphalt content. These data have been plotted (Fig. 43) in a form suggested by Smith (*148*) as a possibility for defining the failure envelope for polymers in tension. Although there is considerable scatter, the failure envelope for this asphaltic concrete in tension can be ap-

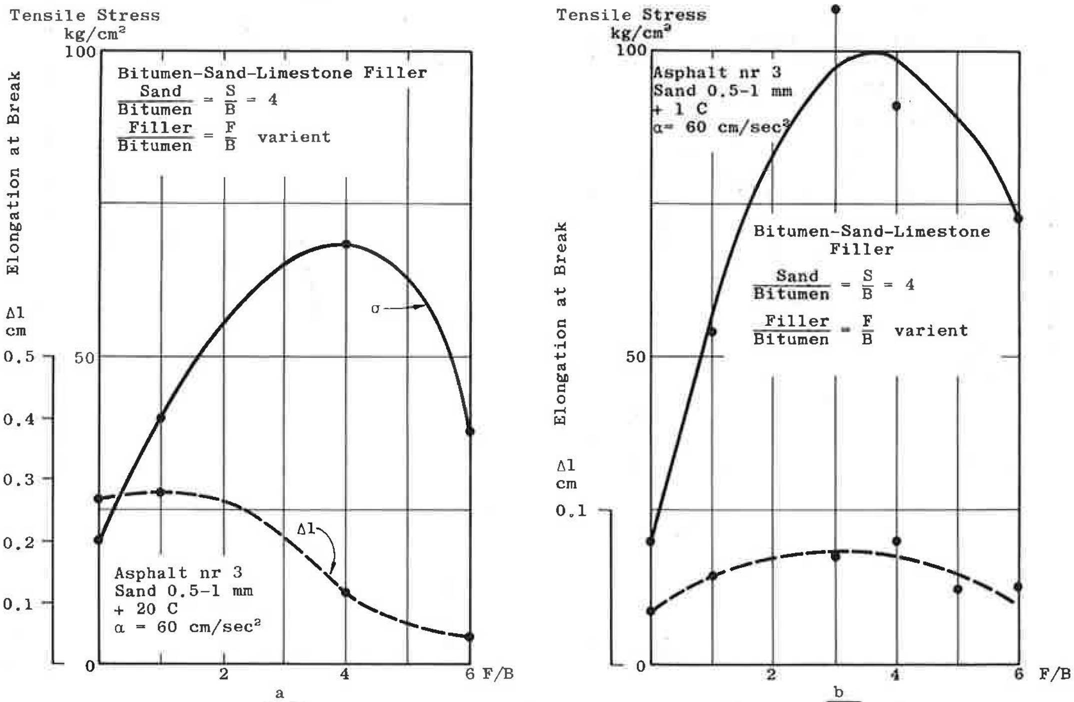


Figure 40. Variation of strength and elongation before break with percentage of filler for mixtures of filler, sand, and asphalt (nr 3); (a) +20 C, (b) +1 C (after Eriksson, 34).

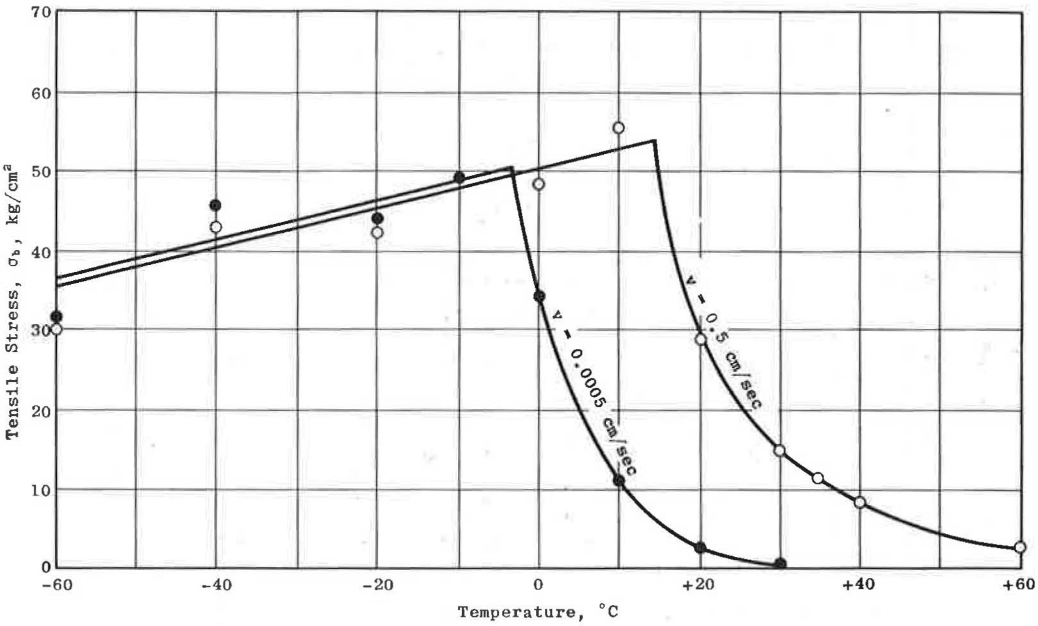


Figure 41. Relaxation between tensile strength (σ_b = maximum of pulling force/cross-sectional area) and temperature (t°C) at constant rate of elongation (v = 0.0005 and 0.5 cm/sec) (after Ericksson, 35).

proximately defined in terms similar to those for polymers. Considerable scatter is not unusual when examining the tensile strength characteristics of asphaltic concrete. Eriksson's data (*35*) shown in Figure 44 illustrate similar scatter. In Figure 43, it appears that the peak tensile strength is of the order of 40 kg/cm² (600 psi). This value corresponds to rapid rates of loading and/or low temperatures (direction indicated by arrow in Fig. 43). It is also interesting to compare the strain at break for this mixture with the data presented by Heukelom and Klomp and summarized in Table 8. The lowest values obtained at low temperatures, as shown in Figure 43, are of the order of 1,000 to 1,200 $\times$ 10^{-6} in. per inch. This range compares favorably with the value for strain of 1,100 $\times$ 10^{-6} in. per inch at — 10 C presented by Heukelom and Klomp (*50*).

The British Road Research Laboratory (*126*) has presented tensile creep data developed for tar-filler mixtures as shown in Figure 45. This figure, as well as Figure 46, which presents data developed by Lee and Markwick (*80*) for a sheet asphalt, shows that there appears to be an optimum asphalt content for mixtures subjected to creep in tension. At low asphalt (tar) contents, comparatively small deformations result in fracture at short loading times; but as the binder content increases, the deformation curves appear to reach a steady creep rate or to level out for sustained periods of time. At still higher asphalt contents, fracture again occurs in comparatively short periods of time, although the strain at break is larger than that at low asphalt contents. Data of this type suggest that properties other than mixture stability, such as fracture strength, can also be optimized through testing. Quite often in the past, stability has been the major concern, and rightly so. However, at

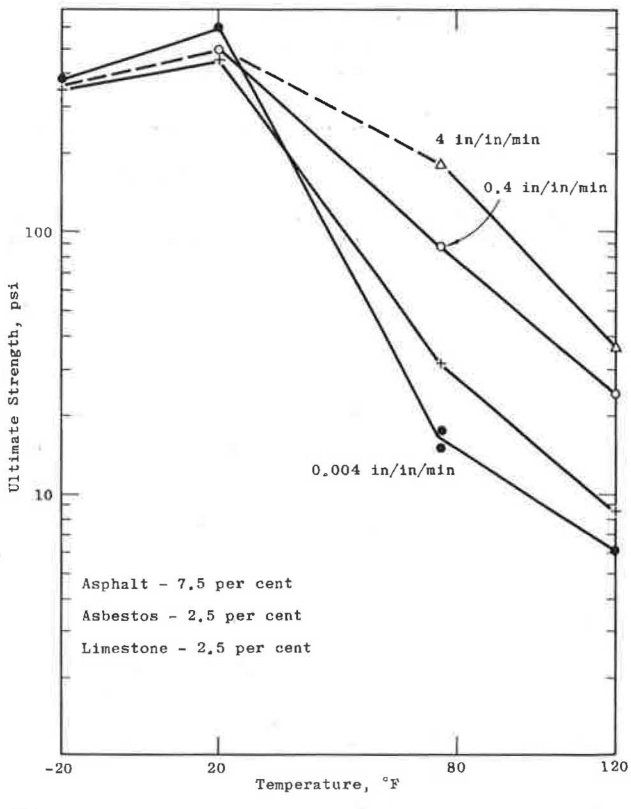


Figure 42. Tensile strength as a function of rate of strain and temperature (after Tons and Krokosky (151) by permission).

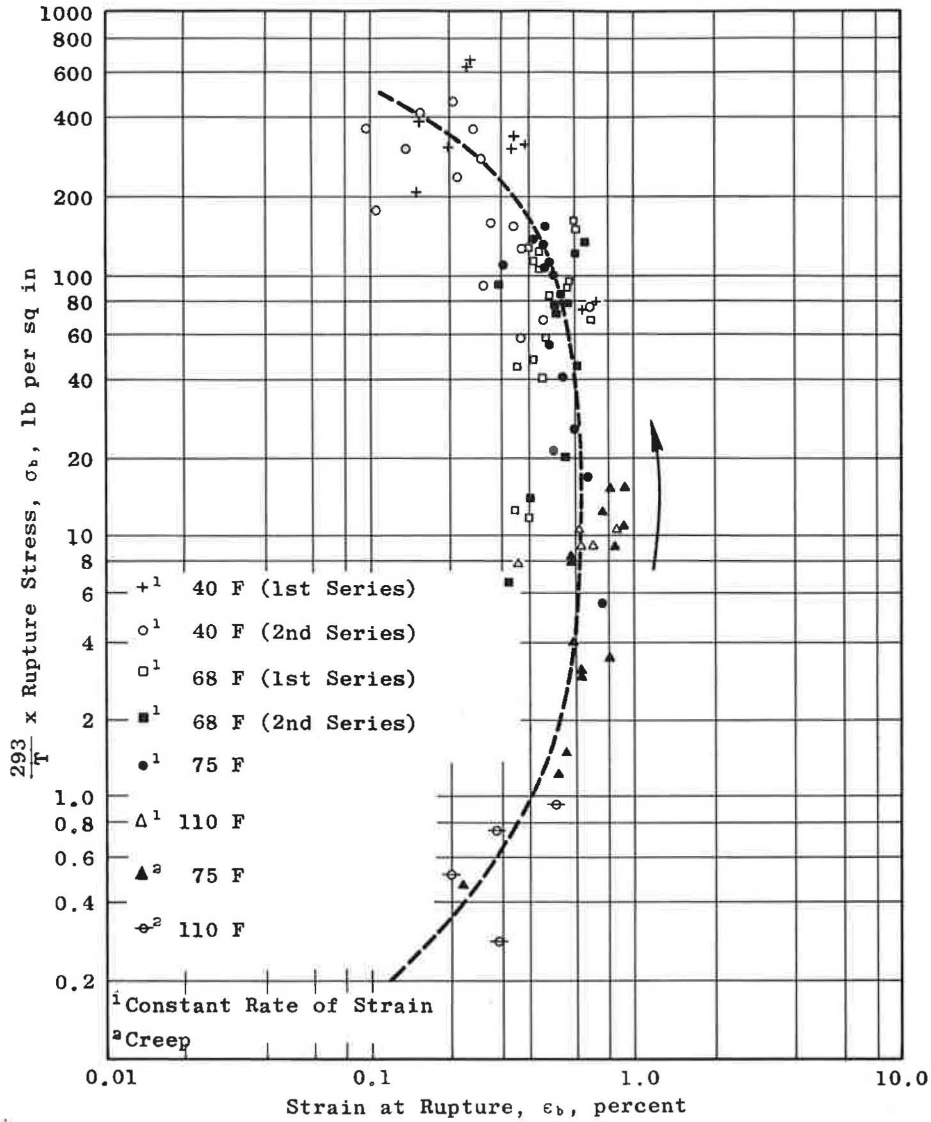


Figure 43. Influence of test method, time of loading, and temperature on the stress-at-break vs the strain-at-break relationship for asphaltic concrete (after Monismith et al. (104) by permission).

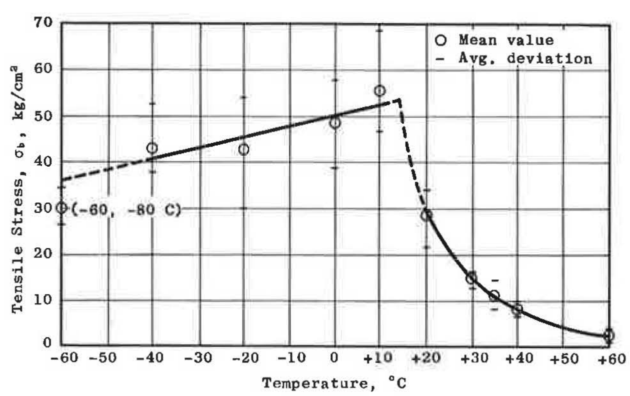


Figure 44. Mean values and average deviations in series of tests on tensile strength at different temperatures and constant elongation rate (after Eriksson, 35).

times, design of mixtures for special application may require consideration of properties in addition to stability. These data illustrate a possible approach to such special cases.

As noted earlier, Mack has suggested that in laboratory tests of asphalts in thin films, the tensile strength does not exceed values of the order of 14 kg/cm² (≈ 200 psi). He attributed these comparatively low values to the influence of high residual stresses, which result from the cooling process of asphalt films on aggregate surfaces because aggregate has a different coefficient of expansion from asphalt. Recently Mack (*87*) has also suggested that, through thermodynamic considerations, one might formulate a mix design to minimize the development of these high residual tensile stresses and thereby reduce the tendency of mixes to fracture. If an asphaltic mix is assumed to consist of coarse

TABLE 8

STIFFNESS AND BREAKING STRENGTH OF ASPHALT-BOUND BASE MATERIALS [a]

MIXTURE COMPOSITION	TEMP. (°C)	DYNAMIC MODULUS (KG/CM^2)	BREAKING STRENGTH (KG/CM^2)	STRAIN AT BREAK ($IN./IN. \times 10^{-6}$)
Gravel, sand, and 50 pen. asphalt cement	+10	66,000	95	2,000
Gravel, sand, and 70 pen. asphalt cement	+10	57,000	95	2,100
Gravel, sand, and 90 pen. asphalt cement	+10	50,000	100	2,700
Gravel, sand, and 110 pen. asphalt cement	+10	36,000	90	7,500
Gravel, sand, and 90 pen. asphalt cement	−10	125,000	75	1,100
Gravel, sand, and 90 pen. asphalt cement	0	85,000	90	1,400
Gravel, sand, and 90 pen. asphalt cement	+10	50,000	100	2,700
Gravel, sand, and 90 pen. asphalt cement	+20	23,000	85	9,000
Gravel, sand, and 90 pen. asphalt cement	+30	10,000	65	13,000
100% sand, and 90 pen. asphalt cement	+10	50,000	85	2,700
60% sand, 40% gravel, and 90 pen. asphalt cement	+10	70,000	80	2,300
40% sand, 60% gravel, and 90 pen. asphalt cement	+10	80,000	85	2,300

[a] Adapted after Heukelom and Klomp (*50*).

aggregate embedded in a matrix of fine aggregate and asphalt, Mack contends that the pavement will have a minimum of residual stresses when the coarse aggregate fraction has the same internal pressure as the matrix. Accordingly,

$$(P_i f)_3 = (P_i f)_1 + (P_i f)_2 \qquad (15)$$

in which

P_i = internal pressure = $\dfrac{TK_e}{1 + aT}$;

f = volume fractions of the constituents with the indices 1, 2 and 3 referring to the asphalt, fine aggregate, and coarse aggregate, respectively;

T = absolute temperature;

K_e = bulk at atmospheric pressure; and

a = coefficient of cubical expansion.

Considering a mixture of asphalt, fine silica, and coarse limestone, Table 9 summarizes the recommended formulation according to Eq. 15. Although this is merely an example, it does point out an approach which might assist the engineer in formulating special mix designs.

SUMMARY

The available data suggest that fracture strength of asphalts in bulk under low-temperature conditions and rapid rates of loading is in the range of 20 to 40 kg/cm² (≈ 290-580 psi). Also, some data reported by Rigden and Lee indicate

TABLE 9

MIX DESIGN BY THERMODYNAMIC METHOD

MATERIAL	COEF. OF CUBICAL EXPANSION	BULK COEFF., K_e	RECOMMENDED PROP. (%) VOLUME	RECOMMENDED PROP. (%) WEIGHT
Limestone	9.0×10^{-6}	1.6×10^{6}	52.6	57.4
Silica	5.5×10^{-6}	3.8×10^{6}	34.0	27.1
Asphalt [a]	6.0×10^{-4}	2.3×10^{4}	13.4	5.5

[a] $G_{aggr} = 2.65$; $G_{asph} = 1.0$.

that weathering does not alter this maximum value. These data also indicate that asphalt type (as influenced by chemical composition) will influence the range of temperature and time where brittle fracture, as opposed to tensile failure with flow, will occur.

Because of these data, both van der Poel and the British Road Research Laboratory suggest that it may be possible to obtain a comparison of the susceptibility of bituminous materials to brittle fracture by measurement of their stiffness.

Although this evaluation appears to be applicable to asphalts in bulk, Mack has indicated that residual stresses may develop in thin films of asphalt upon cooling because of the difference in coefficient of expansion of asphalt and aggregate. These residual stresses are sufficient, he contends, to limit the fracture strength of asphalt to between 14 and 15 kg/cm² (≈ 200 psi), a value much lower than that indicated by van der Poel et al.

The addition of mineral filler to asphalt appears to increase its fracture strength. As indicated by Eriksson, this may be due to the ability of the filler to reduce the sensitivity of asphalt to stress concentrations.

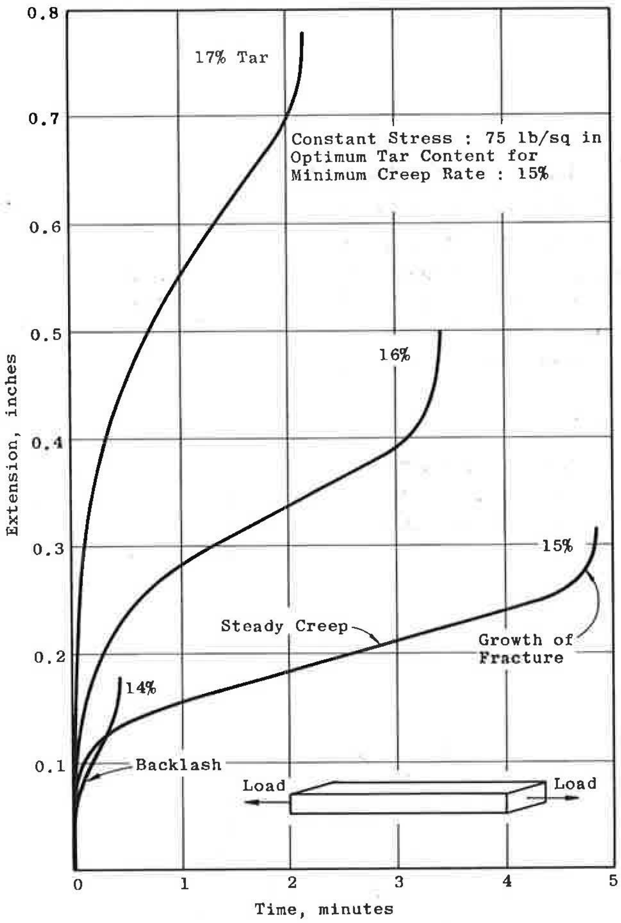


Figure 45. Tensile flow of limestone filler/tar mixtures, showing backlash, steady creep, and onset of fracture (after 126).

For mixtures of asphalt and aggregate, the fracture (tensile) strength under rapid loading and/or low-temperature conditions is in the range of 40 to 100 kg/cm² (≈ 550 to 1,400 psi). Strain at fracture under these conditions is probably of the order of 1,000 to 1,200 $\times$ 10^{-6} in. per inch. Moreover, the sress-strain characteristics for these conditions may be linear to failure. Eriksson has indicated that the slope of the stress-strain curves of his mixtures under these conditions approached the dynamic stiffness of the material. Thus, as with the asphalt, stiffness may be a good criterion for determining the susceptibility of material to brittle fracture.

No data are available at this time for asphaltic concrete which would indicate the influence of weathering on the magnitude of either the stress or strain at break, because all of the reported values are for mixtures tested immediately after preparation.

DESIGN IMPLICATIONS

As a factor in the design of asphaltic surfacings, fracture or tensile strength appears to be of importance in three areas of design: (a) for failure under single load applications, (b) for pavement slippage wherein tensile strength would be an important consideration, and (c) for thermal stresses as discussed in Chapter Four.

On the basis of research performed to date, the general order of magnitude of the fracture strength appears to range between 500 and 1,400 psi, depending on the temperature and the rate of loading with a limiting strain of approximately 1.0 $\times$ 10^{-3} in. per inch at relatively low temperatures. This strain is approximately one order of magnitude greater than allowable for acceptable levels of strain in fatigue loading.

Because use of the multilayered theory enables the engineer to compute the stress and strain for any given loading condition, including considerations for time and temperature, it should be possible to estimate with reasonable accuracy the limiting maximum load to which a surfacing can be subjected without failure.

It has been suggested that the greatest risk of fracture will occur at low temperatures (− 5 C) and that stress is the factor of prime concern. For typical asphaltic mixes with 85- to 100-penetration asphalt, a tensile strength of approximately 1,100 psi may be expected; however, Dormon indicates that, after the application of repetitive loads, this tensile strength should be reduced 25 percent, or to 850 psi. Under conditions of heavy loading, he suggests using a tensile strength of 700 psi and that only a limited number of loadings should be allowed which would cause a stress of this magnitude. Peattie (*115*) has suggested limiting tensile stress to 900 psi, probably at the colder temperatures. The use of this criterion would appear to be particularly applicable under conditions of subgrade thawing during periods of cold temperature.

The tensile strength of asphaltic concrete may play a key role in preventing slippage cracks in asphaltic layers, particularly in the case of thin overlays subjected to tractive forces associated with acceleration or deceleration or special applications, such as the case of thin surfacings on

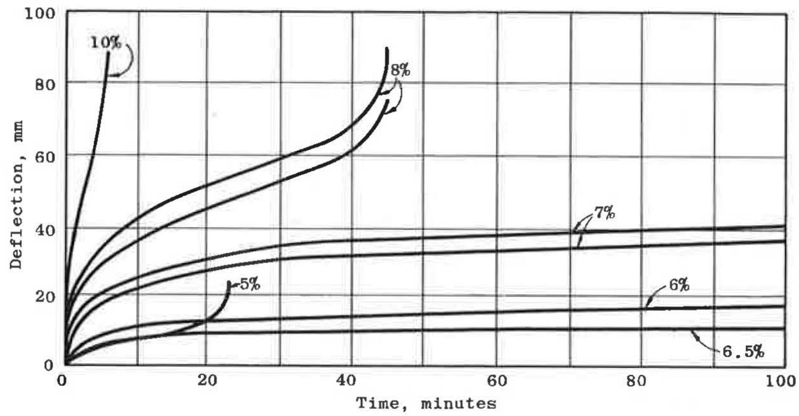


Figure 46. Typical deflection-time curves showing effect of binder content (after Lee and Markwick, 80). Aggregate: 1/4- to 3/16-in. Mountsorrel granite, 60.6%; 25 to 36 B.S.S. Mountsorrel granite, 21.2%; portland cement filler, 18.2%. Binder: 300-pen. horizontal retort pitch. Temperature: 25C.

orthotropic bridge decks. Further consideration is given to this problem in Chapter Six.

Thermal stresses are discussed in Chapter Four. It is generally concluded that, for all but the most severe cases, thermal stresses *per se* are not likely to exceed those of fracture strength. However, superposition of tensile stress due to wheel loads, in addition to the thermal stress, could conceivably exceed the fracture strength. Stresses due to wheel load can be computed from the multilayered theory as illustrated in Chapter Five.

To increase the tensile strength of asphalt and asphaltic concrete, research has indicated mineral fillers to be effective. The use of lime or asbestos (*151*) has been shown to increase the viscosity of the asphalt binder, hence can be reasoned to increase the resistance of the surfacing to tractive forces which may cause tensile cracks in thin overlays.

Tensile strength may also be used as a means to evaluate the layer equivalency of various asphaltic surfacings. Hveem and Sherman (*64*) have used cohesion (tensile strength) as a factor for evaluating the relative structural capability of asphaltic concrete to untreated layers of base and subbase. With further effort, it may be possible to measure tensile strength and establish a more definitive means of estimating layer equivalency of asphaltic concrete, giving credit to those mixes designed to produce higher tensile strengths. For example, on the basis of tests performed on bulk asphalt, it appears that the optimum film thickness for maximum tensile strength is something in excess of 10 microns. Campen (*20*) and Goode and Lufsey (*41*) have indicated that mixes produced by current mix design methods result in film thicknesses of 7 to 10 microns. This seems to indicate that increased film thicknesses would result in higher tensile strength. As with all design factors, however, a balance must be achieved. Therefore, care must be taken to satisfy all of the design requirements while achieving an optimum tensile strength.

CHAPTER FOUR

THERMAL STRESSES

Asphaltic mixtures, like other engineering materials, undergo volume changes with changes in temperature. If these volume changes are restricted because of constraints such as friction between the pavement and the underlying layer or because of differential temperature changes in the material itself, it is possible that stresses will develop which may be of sufficient magnitude to cause cracking of the pavement. If the thermal stresses are not of sufficient magnitude in themselves, they may be additive to other stresses such as those resulting from vehicle loads, which in turn could lead to cracking. Thus, under certain circumstances it may be worthwhile to make an estimate of these temperature-induced stresses resulting from restriction of volume changes to aid the engineer in the proper design of the asphaltic structure.

DEFINITION OF TERMS

To estimate temperature stresses, certain physical constants are necessary. In this section, these physical characteristics are introduced in a general way. Brief definitions are provided and commonly used units are assigned, as follows:

1. The coefficient of linear expansion gives a measure of the change in length of material over a particular temperature range and can be defined by

$$a = \frac{\Delta L}{L_0} \frac{1}{\Delta t} \tag{16}$$

in which

a = coefficient of linear expansion, in in./in./°F, or cm/cm/°C;
L_0 = length at some reference temperature; and
ΔL = change in length due to temperature change, Δt, from reference temperature.

2. Similarly, the cubical coefficient of expansion can be defined by

$$\beta = \frac{\Delta V}{V_0} \frac{1}{\Delta t} \tag{17}$$

in which

β = cubical coefficient of expansion, in.³/in.³/°F, or cm³/cm³/°C;
V_0 = volume at some reference temperature; and
ΔV = change in volume due to temperature change, Δt, from reference temperature.

Normally, β can be considered to be equal to $3a$.

3. The specific heat capacity is, for all practical purposes, numerically equal to the quantity of heat which must be supplied to a unit mass of material to increase its temperature 1°; i.e.,

$$C = \frac{Q}{m\,\Delta t} \tag{18}$$

in which

C = specific heat capacity, cal/gm°C, or Btu/16°F;
Q = quantity of heat, cal or Btu; and
m = mass of material, gm or lb.

Strictly speaking, the specific heat capacity is not a constant but is a function of temperature. From an engineering standpoint, however, the specific heat capacity is often reported as constant for a particular temperature range.

4. The capacity of a material to transfer heat is termed its heat or thermal conductivity. In the steady state, the rate of heat flow is proportional to the area and to the temperature difference between the faces of the slab and inversely proportional to the thickness. This heat transfer is usually given by

$$\frac{dQ}{dt} = -k \frac{dt}{dx}\, dy\, dz \tag{19}$$

in which

k = coefficient of thermal conductivity, or simply thermal conductivity;
$\frac{dQ}{dt}$ = rate of heat transfer;
$\frac{dt}{dx}$ = temperature gradient; and
$dy\,dz$ = area.

It should be noted that the thermal conductivity is also expressed in many other units; however, the two systems given are fairly common in engineering usage.

Some available information on these characteristics for the components, as well as for the mixture itself, are summarized in the next section.

TABLE 10

CUBICAL COEFFICIENT OF EXPANSION OF ASPHALTS [a]

ASPHALT SOURCE	PENETRATION 77 F, 5 SEC, 100 GR (DMM)	RING-AND-BALL SOFT POINT (°C)	β, PER °C ($\times 10^{-4}$)	TEMP. RANGE (°C)
Borneo	47	47	6.1	30–60
California (residual)	54	48	6.1	25–65
Mexico (residual)	55	55	6.2	25–65
Venezuela (residual)	44	55	6.0	25–65
Venezuela (blown)	37	68	6.2	25–65
Venezuela (blown)	35	87	6.1	30–90

[a] Adapted after Saal (*131*).

TABLE 11

CUBICAL COEFFICIENT OF EXPANSION FOR MEXICAN RESIDUAL ASPHALTS [a]

PENETRATION, 77 F, 5 SEC, 100 GR (DMM)	RING-AND-BALL SOFT POINT (°C)	β, PER °C ($\times 10^{-4}$)
196	38.5	6.0
65	50	6.1
65	52	6.1
50	53	6.1
45	56	6.1
23	69	6.2

[a] Adapted after Saal (*131*).
[b] Temperature range 15 to 200 C.

THERMAL PROPERTIES

Asphalts

The most extensive studies of the thermal characteristics of asphalts have been conducted by the Royal Dutch Shell investigators (*122*). For this reason, the data developed by Saal (*131*), primarily, are presented to illustrate the range in values to be expected for the thermal properties of asphalts.

Table 10 presents data for the cubical coefficient of expansion for a variety of asphalts, and Table 11 illustrates the influence of the hardness of asphalt on this coefficient. In general, the data indicate that the majority of asphalts have a coefficient of about 6×10^{-4} per °C, although it appears to increase slightly as the hardness of the asphalt increases.

This value of the cubical coefficient is applicable to asphalts which are considered to be in the fluid state; when they change to solids with reduction in temperature, a different coefficient of expansion is applicable. A schematic diagram of the change in specific volume with change in temperature for an asphalt is shown in Figure 47. Above the temperature termed the glass transition temperature, T_g, the coefficients given in Tables 10 and 11 are applicable. Below T_g, a lesser value in the range $2\text{-}4 \times 10^{-4}$/°C appears to be representative (*134, 164*) and is somewhat dependent on asphalt composition, as given in Table 12. Values for T_g for a series of asphalts over a range in viscosities developed by Schmidt et al. (*134*) are presented in Figure 48.

From an engineering standpoint, T_g has practical significance in that below this temperature the asphalt behaves as an elastic material regardless of the time of loading, whereas above T_g time or rate of loading becomes significant in that the material exhibits viscoelastic response.

Values for specific heat capacity for a series of asphalts over the temperature range 0 to 300 C presented in Table 13 were obtained from data presented by Saal (*132*). These data indicate that the specific heat increases linearly with temperature and can be expressed by an equation of the form suggested by Mack (*86*), as follows:

$$c = c_0 + C_1 \,\Delta T \qquad (20)$$

in which

c = specific heat at particular temperature;
c_0 = specific heat at reference temperature (e.g., 0 C); and
C_1 = constant (last column of Table 13).

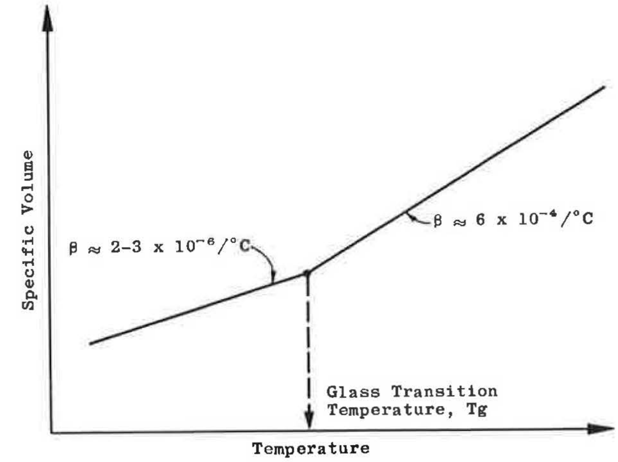


Figure 47. Volume-temperature relationship for asphalt, illustrating transition from solid to fluid state.

For most asphalts, a value of about 0.4 cal/gm at 0 C appears to be reasonable.

A summary of thermal conductivities obtained for a variety of asphalts is presented in Table 14. These data were also obtained from Saal (*131*). It appears that a value for thermal conductivity of approximately 0.13 kg cal m/m²,°C,hr is representative of most of the asphalts. In general, the thermal conductivity decreases slightly with temperature and varies in the range of 0 to 70 C according to (*86*)

$$k = k_0(1 - C_2 \,\Delta t) \qquad (21)$$

in which

k = coefficient of thermal conductivity at particular temperature;
k_0 = coefficient of thermal conductivity at reference temperature (e.g., 0 C); and
C_2 = constant (last column of Table 14).

TABLE 12

INFLUENCE OF CHEMICAL COMPOSITION OF ASPHALT ON GLASS TRANSITION TEMPERATURE AND COEFFICIENT OF CUBICAL EXPANSION [a]

SAMPLE	ASPHALTENE CONTENT (% WT)	DENSITY 20 C (MG/CM³)	T_g (°C)	PER °C β (× 10⁻⁴) ABOVE T_g	PER °C β (× 10⁻⁴) BELOW T_g
A. Asphaltenes	100	1.079	None	—	—
B. Mixture of samples A and I, 3 to 1 proportions	75	1.053	None	—	—
C. Blown asphalt	62	1.039	2	5.8	3.7
D. Blown asphalt	59	1.034	0	6.0	3.8
E. Blown asphalt	57	1.030	−2	6.3	4.0
F. Blown asphalt	52	1.027	−6.5	6.6	3.9
G. Blown asphalt	51	1.026	−7.5	6.8	3.9
H. Straight run asphalt	29	1.014	−22.5	6.9	3.7
I. Maltenes	0	1.004	−37.5	7.6	3.4

[a] Adapted after Wada and Hirosi (*164*).

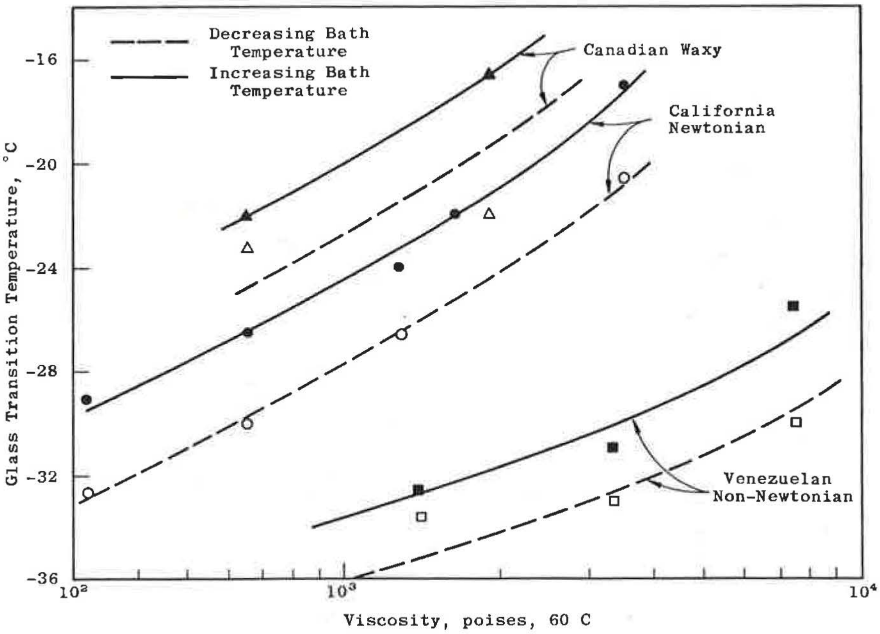


Figure 48. Glass transition temperature of various asphalts (after Schmidt, Boynton and Santucci, 134).

Aggregates

In the field of concrete technology in conjunction with studies of mass concrete, a large number of investigations have been conducted in the thermal expansion characteristics of aggregates; e.g., Troxell and Davis (*155*) give a summary of, and additional references to, these studies. They report average values for the coefficient of linear expansion ranging from 7×10^{-6} per °F for quartz, to less than 3×10^{-6} per °F for some limestones. Mitchell's (*92*) linear expansion data for 41 aggregate minerals are summarized in Table 15.

Blanks and Kennedy (*59*) indicate that many mineral crystals and rock particles show different thermal coefficients of expansion in different directions. Thus, the values

TABLE 13

SPECIFIC HEAT CAPACITY OF ASPHALTS [a,b]

ASPHALT SOURCE	PENETRATION, 77 F, 5 SEC, 100 GR (DMM)	RING-AND-BALL SOFT POINT (°C)	SPECIFIC HEAT CAPACITY (CAL/GM °C) [c] 0 C	100 C	200 C	300 C	CHANGE IN SPECIFIC HEAT PER 1 C ($\times 10^{-4}$)
Venezuela (residual)	177	40	0.425	0.472	0.520	0.567	4.7
Venezuela (residual)	23	63	0.409	0.463	0.518	0.572	5.4
Venezuela (residual)	7	97	0.382	0.455	0.527	0.600	7.3
Mexico (residual)	23	65	0.429	0.462	0.499	0.534	3.5
Mexico (residual)	39	85	0.430	0.462	0.494	0.526	3.2
Mexico (residual)	25	87	0.402	0.458	0.514	0.570	5.6
Borneo (residual)	25	53	0.378	0.456	0.534	0.612	7.8
Highly cracked material	2	73	0.419	0.459	0.499	0.538	4.0

[a] Adapted after Saal (*132*).
[b] Asphalts free from paraffin wax.
[c] 1 cal/gm°C = 1 Btu/lb°F.

TABLE 14

THERMAL CONDUCTIVITY OF ASPHALTS [a]

ASPHALT SOURCE	PENETRATION 77 F, 5 SEC, 100 GR (DMM)	RING-AND-BALL SOFT POINT (°C)	THERMAL CONDUCTIVITY, k (KG CAL M/M^2,°C,HR) [b] 0 C	20 C	40 C	60 C	70 C	COEFF. C_2, EQ. 20 ($\times 10^{-4}$)
Venezuela (residual)	177	40	0.136	0.133	0.130	—	—	11.0
Venezuela (residual)	23	63	0.141	0.137	0.133	0.129	0.125	16.2
Venezuela (residual)	7	97	0.141	0.137	0.130	0.124	0.127	16.7
Mexico (residual)	23	65	0.137	0.135	0.133	0.131	—	7.3
Mexico (residual)	39	85	0.141	0.139	0.138	0.136	0.134	7.1
Mexico (residual)	25	87	0.150	0.146	0.143	0.140	0.136	13.3
Bornea (residual)	25	53	0.120	0.117	0.115	—	—	10.4
Highly cracked material	2	73	—	0.129	0.126	0.124	0.121	12.4

[a] Adapted after Saal (*131*).
[b] 1 kg cal m/m^2, °C,hr = 0.67 Btu ft/ft^2, °F,hr.

given in Table 15 should be considered to be representative overall values for linear expansion.

Comparing the values of thermal expansion for aggregates with those of the asphalts, it can be seen that there is at least an order of magnitude difference in the expansion characteristics of these materials.

Hogbin (*59*) has presented data on the specific heat capacities of various aggregates from sources in Great Britain. His results are given in Table 16. It will be noted that these values are approximately one-half the values listed for asphalts in Table 13.

Thermal conductivity values for rocks are given in Table 17. These values correspond approximately to the range for thermal conductivities of aggregates suggested by Saal (*131*); namely, 1 to 2 kg cal m/m^2, °C, hr ($\approx$ 0.7 to 1.4 Btu ft/ft^2,°F,hr). Unfortunately, there does not appear to be a large amount of information available on aggregates *per se*. In the field of mass concrete research, for example, the influence of the aggregate type on the thermal conductivity of concrete has been investigated extensively by studies of the concretes containing different aggregates (*17*). However, even though this research is extensive, no data on the conductivity characteristics of the aggregates themselves are presented. From the data presented in Table 17, however, it appears that the conductivity of the aggregate is approximately one order of magnitude larger than that of the asphalt.

Asphaltic Mixtures

The coefficients of linear and cubical expansion of asphaltic concrete have been investigated by Hooks and Goetz (*60*). Their investigation included the effects of a number of mix variables; namely, aggregate type (limestone and gravel-sand-limestone filler), asphalt hardness (60-70 and 85-100

TABLE 15

COEFFICIENT OF LINEAR EXPANSION OF AGGREGATES [a]

MATERIAL	LINEAR EXPANSION, α (IN./IN. $\times 10^{-6}$/°F)
Limestone	1.2 to 5.3
Sandstone	3.7 to 6.5
Quartzite	4.5 to 5.6
Quartz	5.6 to 8.0
Basalt	3.4 to 4.1
Granite	2.0 to 4.5

[a] Adapted after Mitchell (*92*).

TABLE 16

SPECIFIC HEAT CAPACITIES OF OVEN DRIED ROAD MATERIALS FOR THE TEMPERATURE RANGE 0 TO 26 C [a]

MATERIAL	PARTICLE SIZE	MEAN SPECIFIC HEAT (CAL/GM, °C)
Flint gravel	⅜ in.	0.182
Granite	⅜ in.	0.194
Olivine dolerite	⅜ in.	0.189
Limestone	Dust	0.190
Sand	Fine	0.170

[a] Adapted after Hogbin (*59*).

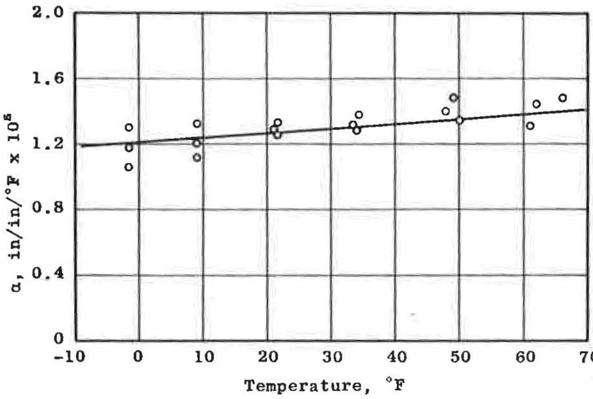


Figure 49. Influence of temperature on the coefficient of thermal expansion, α, for 1 × 1 × 12-in. specimens of asphaltic concrete ($\gamma = 152$ pcf) (after Monismith, Secor and Secor (102) *by permission).*

TABLE 17

THERMAL CONDUCTIVITIES OF ROCKS

MATERIAL	UNIT WEIGHT (PCF)	TEMP. (°F)	SPEC. HEAT (BTU FT/FT², °F,HR)	REF.
Sandstone	140	104	1.06	(*117*)
Granite	—	—	1-2.3	(*117*)
Calcium carbonate	162	86	1.3	(*117*)
Limestone [a] (15% by vol. water)	103	73	0.5	(*117*)
Limestone	—	—	0.3-0.75	(*89*)
Various aggregates	—	—	0.7-1.4	(*131*)

[a] 15 percent (by volume) water.

penetration asphalt cements), and asphalt content (4.0, 4.5, 5.0, and 5.5 percent by weight of aggregate). One aggregate gradation utilized conformed to gradation 3b of the Corps of Engineers, U. S. Army Specifications (¾-in. maximum size and 3.5 to 7.5 percent passing the No. 200 sieve). On the basis of both linear and volumetric measurements, they concluded that increased asphalt content resulted in an increase in both the linear and cubical coefficients of expansion. Little difference was noted between mixtures containing the 60-70 and 85-100 penetration asphalt cements. In addition, mixtures containing limestone and those containing the gravel also produced about the same values, presumably due to the comparatively large contributions of the asphalt to the coefficients of expansion. They suggest that overall average values for the mixtures investigated are 2.0×10^{-5} per °C for the linear coefficient and 7.0×10^{-5} per °C for the cubical coefficient of expansion. With the same volumetric apparatus they report a value of 2.3×10^{-5} per °C for the limestone aggregate alone.

TABLE 18

SPECIFIC HEAT CAPACITIES OF PAVEMENT COURSES (DRY) FOR TEMPERATURE RANGE OF 0 TO 26 C [a]

COURSE	COMPOSITION: MATERIALS	COMPOSITION: (%)	HEAT CAPACITY (CAL/GM °C)
Wearing	40-50 pen. asph.	7.3	
	Limestone	8.2	
	Granite	36.3	
	Sand	48.2	0.199
Sand carpet	Asphalt	11.0	
	Sand	76.0	
	Limestone	13.0	0.200
Concrete, base	Flint	54.7	
	Sand	33.5	
	Cement	11.8	0.181
Base	200 pen. asph.	4.75	
	Flint	95.25	0.194
Wearing, exper. slab	40-50 pen. asph.	8.3	
	Limestone	9.6	
	Granite	25.0	
	Sand	57.1	0.199
Base, exper. slab	40-50 pen. asph.	5.7	
	Granite	65.0	
	Sand	29.3	0.200
Mastic asphalt (possible construction of high heat capacity)	Asphalt	12.0	
	Granite	25.0	
	Limestone	63.0	0.219

[a] Adapted after Hogbin (*59*).

Monismith et al. (*103*) have presented some data for the coefficient of linear expansion for an asphaltic concrete containing an 85-100 penetration asphalt cement and a dense-graded granitic aggregate. Their data are shown in Figure 49. A linear expansion value of 1.2 to 1.4×10^{-4} is indicated in the temperature range − 10 to + 70 F. This compares reasonably well with the data reported by Hooks and Goetz (*60*).

Although few data are available on the specific heat of mixtures, Abraham (*2*) has suggested the following for this determination when the specific heats of the asphalt and aggregate are known:

$$C_{mix} = 0.01 \left[(100 - x)\, C_{asph} + x\, C_{aggr}\right] \qquad (22)$$

in which

C_{mix} = specific heat capacity of the mixture;
C_{asph} = specific heat capacity of the asphalt;
C_{aggr} = specific heat capacity of the aggregate; and
x = percent by weight of aggregate.

Hogbin (*59*), using a method developed by Marsh (*88*) (whose equation is essentially the same as Eq. 22), has computed the heat capacities of a series of courses employed in the British Road Research Laboratory's experimental heated roads. These values (Table 18) are similar to the aggregate data given in Table 16. This would follow also for Eq. 22.

As with the other thermal properties, the thermal conductivity of mixtures does not appear to be well defined. Barber (*11*) has suggested and used a value of 0.7 Btu ft/ft², °F,hr,

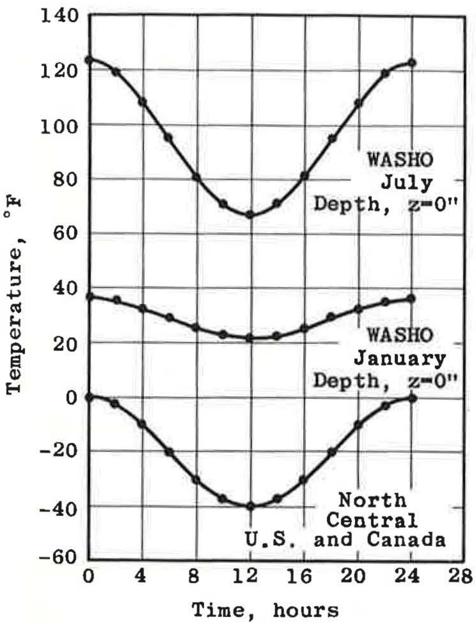


Figure 50. Assumed temperature-time relationship (after Monismith, Secor and Secor (102) *by permission.*

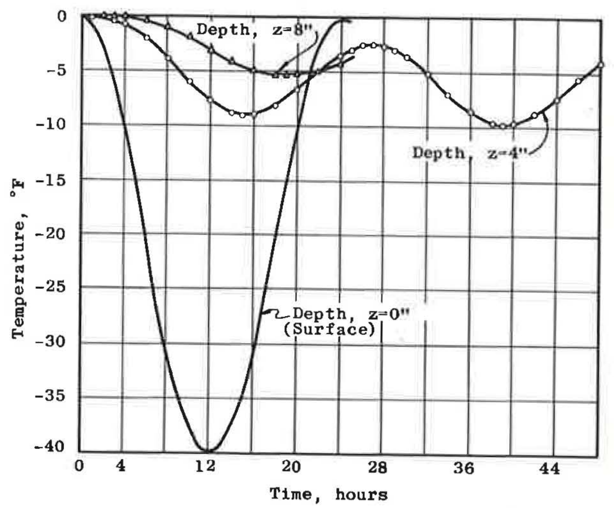


Figure 51. Temperature vs time relationship at various depth within slab for 0 — 40 F temperature variation at surface (after Monismith, Secor and Secor (102) *by permission).*

in computations of pavement temperatures. Saal (*131*) has suggested that thermal conductivity of a mix can be determined approximately from

$$\log k_{\text{mix}} = X' \log k_{\text{asph}} + (1\text{-}X') \log k_{\text{aggr}} \qquad (23)$$

in which

k_{mix} = thermal conductivity of the mixture;
k_{asph} = thermal conductivity of the asphalt;
k_{aggr} = thermal conductivity of the aggregate; and
X' = fraction by volume of asphalt.

Using his values for the thermal conductivities of both the asphalt (0.13 kg cal m/m^2,°C,hr, Table 14) and the aggregate (1.5 kg cal m/m^2,°C,hr, Table 17), a value of approximately 0.7 Btu ft/ft^2,°F,hr, is obtained. It appears, however, that this value would be somewhat dependent on the void content of the mixture and that the value determined from Eq. 22 would be applicable to well-compacted (less void content) mixtures.

Temperature Distribution

To determine the temperature distribution in a pavement (which in turn permits stress determination), use can be made of the heat and conduction equation.

Monismith et al. (*103*), using the equation for heat and conduction, determined the temperature distribution with depth in a slab subjected to the surface temperature distributions shown in Figure 50. Results of their computations for the 0 to — 40 F surface variation are shown in Figure 51, and are based on values for thermal conductivity and specific heat of 0.7 Btu ft/ft^2,°F,hr and 0.22 Btu/lb, °F, respectively.

It should be noted that the temperature distributions shown in Figure 51 were developed with a computer solution of the heat conduction equation. Additional information relative to this solution have been given by Monismith (*103*).

That this form of temperature distribution, and particularly the variation with depth shown in Figure 51, is not unreasonable is substantiated by data published by The Asphalt Institute (*7*). Pavement temperature data obtained from measurements in slabs 6 and 12 in. thick are shown in Figure 52. Similar trends are noted in both Figures 51 and 52 as far as attenuation of change with depth and time lags to reach minimum values.

Barber (*11*) has computed maximum surface temperature of pavements using weather report information. He has considered not only the relation between air temperature and pavement temperature but also the influence of wind, precipitation, and solar radiation on this temperature. A comparison between computed and measured surface temperatures for the Hybla Valley Test Road is presented in Figure 53; it will be noted that reasonable agreement is indicated.

From the foregoing data, it would thus appear that pavement temperatures can be reasonably estimated from existing solutions. In the case of the solution presented by Monismith et al., the procedure is probably applicable to thicker sections of asphaltic concrete. For situations where the pavement section is constructed with materials of differing thermal conductivities, additional research appears necessary. The acquisition of substantiating field evidence should not be too difficult, however, particularly if use is made of equipment such as that recently developed by Trott (*154*), which permits continuous recordings of temperatures to be made. This particular apparatus records the durations of each of twelve temperature levels in 5 C steps between — 10 C and + 50 C.

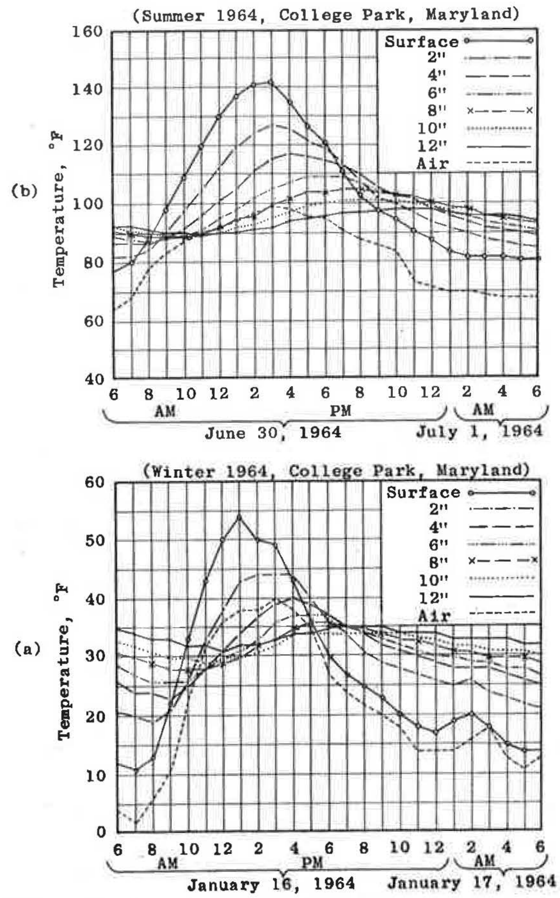


Figure 52. Asphaltic concrete pavement temperatures at different depths (after 7).

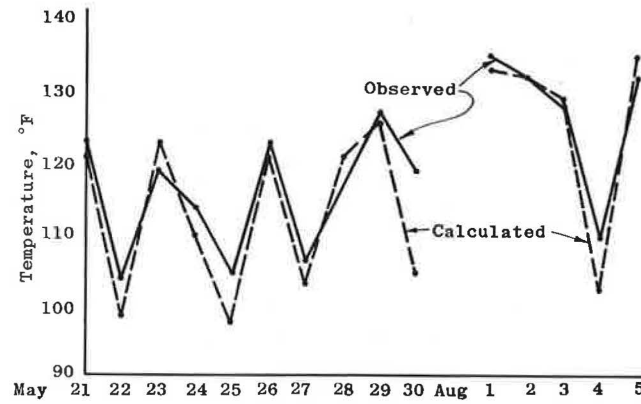


Figure 53. Bituminous pavement surface temperatures, Hybla Valley Test Road (after Barber, 11).

Thermal Stress Determination

Saal (*133*) has utilized the stiffness concept (see Chapter Two) to illustrate the relation of a particular kind of asphalt to minimize thermal stresses. He has suggested an equation of the form:

$$S_{max} = \frac{\sigma_f}{a\ \epsilon} \tag{24}$$

in which

S_{max} = maximum stiffness to preclude cracking from thermal stresses at low temperatures;
σ_f = fracture strength of asphaltic mixture;
ϵ = limiting deformation; and
a = coefficient depending on the state of stress*.

The limiting deformation or strain is obtained from the relation $\epsilon = a\ \Delta T$. Saal's computed values for a limiting stiffness based on Eq. 9 and mixture stiffnesses at a temperature of − 10 C, had a time of loading of 10^4 sec (about 3 hr) and have been summarized in Table 19. For the conditions which he has used†, the stiffnesses of the mixes prepared with all of the asphalts do not exceed his suggested maximum value, 5.1×10^4 kg/cm².

Monismith et al. (*103*), using principles of linear visco-

* For a two-dimensional state of stress and for a material with $\nu = 0.5$, when $\sigma_1 = \sigma_2 (\sigma_3 = 0)$, $a = 2$; when $2\sigma_2 (\sigma_3 = 0)$, $a = 4/3$.
† E.g., ΔT = 10 C (from 0 to − 10 C).

TABLE 19

COMPARISON OF CALCULATED LIMITING STIFFNESS TO MINIMIZE FRACTURE FROM THERMAL STRESS WITH STIFFNESSES OF VARIOUS MIXTURES [a]

MIXTURE	ASPHALT, BY VOL. (%)	TIRE PRESSURE, σ_1 (KG/CM²)	CALC. LIMITING STIFFNESS, S_{max} [b] (KG/CM²)	CONDITIONS FOR MIXTURE STIFFNESS DETERM. TEMP. (°C)	TIME (SEC)	MIXTURE STIFFNESS (KG/CM²) MIX 1 100 PEN.[c] 45 C [d]	MIX 2 100 PEN.[c] 40 C [d]	MIX 3 50 PEN.[c] 45 C [d]	MIX 4 25 PEN.[c] 60 C [d]
Asph. conc.	15	9 [e]	5.1×10^4	−10	10^4	3.1×10^2	5.1×10^2	5.1×10^3	1.0×10^4
Sand-asph.	20	4 [f]	5.1×10^4	−10	10^4	2.0×10^2	3.1×10^2	3.1×10^2	5.1×10^3

[a] Adapted after Saal (*131*). [b] By Eq. 22. [c] Penetration, dmm., at 77F. [d] Ring-and-ball softening point. [e] For very heavy vehicles. [f] For lighter vehicles.

elasticity and the creep compliance data shown in Figure 14, have computed by numerical methods the stresses at the surface of an asphaltic concrete slab subjected to the temperature distributions shown in Figure 50. These computations are shown in Figure 54. It will be noted that for the summer and winter conditions at the WASHO Test Road, the temperature-induced stresses are comparatively low. During the summer the computed tensile stress is less than 50 psi and in winter less than about 150 psi. Surface stresses resulting from the 0 to − 40 F distributions reach a peak value of about 3,300 psi. This stress considerably exceeds the maximum fracture strength of approximately 1,400 psi for all of the mixtures reported in Chapter Three. Thus, under these circumstances, it is not difficult to visualize the possibility of crack formation.

Although these authors indicate that their method of stress computation may produce computed stresses that are probably higher than those which will occur in an actual pavement, because they have assumed the paving slab to be of infinite extent in both horizontal directions, they do suggest that when the temperature of the mix drops below 0 F, thermal stresses of sufficient intensity to crack the mix may be developed.

Another point to be considered is that even though the thermal stresses may not be large enough to cause cracking of the mix, they will be additive to the tensile stresses resulting from load, the combination of which may, in turn, lead to cracking.

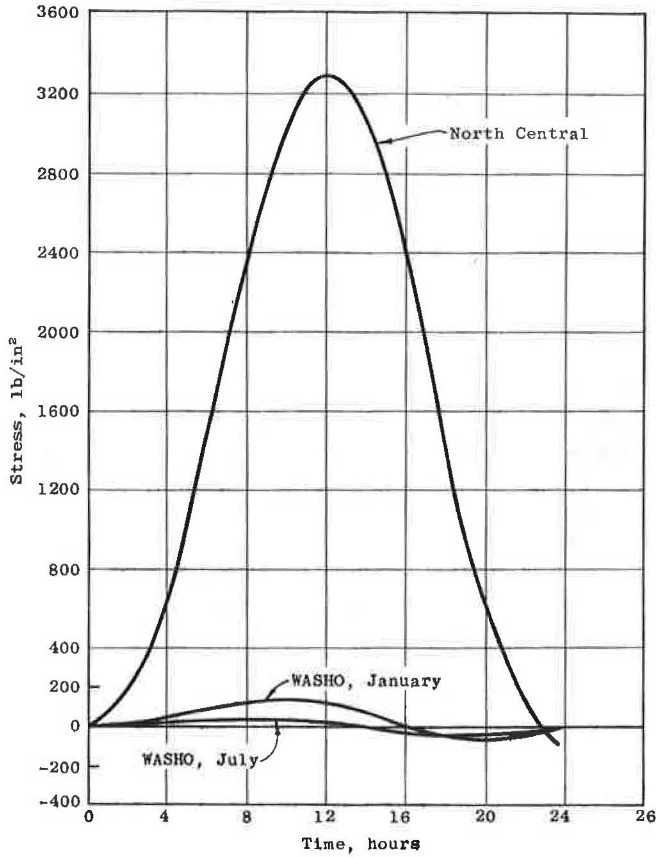


Figure 54. Tensile stress vs time at surface of pavement slab for various temperature conditions (after Monismith, Secor and Secor (102) by permission).

SUMMARY

In this chapter data on thermal properties of asphalts, aggregates, and asphaltic mixtures have been summarized to indicate the characteristics required to determine temperature distributions in mixtures and to determine the magnitude of thermal stresses which might be expected because of restraint of volume changes resulting from changes in temperatures.

Above a characteristic temperature, termed the glass transition temperature, asphalts display a cubical coefficient of expansion of $5\text{-}7 \times 10^{-4}$ per °C. Below this temperature, the coefficient is reduced to $2\text{-}4 \times 10^{-4}$ per °C. With respect to the asphalt, this temperature gives a measure of the transition from elastic behavior to behavior where time effects become important. This, in turn, could have significance with respect to the behavior of paving mixtures, in that asphalts with higher glass transitions may result in mixtures where this transition from time-dependent to elastic behavior occurs at higher temperatures than for mixtures prepared with asphalts with lower transition temperatures. As Monismith et al. have shown, it is primarily in the range where the mixture behaves elastically that high thermal stresses may develop; thus, it is possible that the glass transition temperature of the asphalt will have significance as far as the development of thermal stresses in the mix is concerned.

The data for the coefficient of thermal expansion for mixtures indicate that its magnitude is between that of the aggregate and the asphalt, whose coefficients are at least one order of magnitude different. A value of α of about 2×10^{-5} per °C would appear to be representative of asphaltic concrete, with this value being higher as the asphalt content is increased. The coefficient of cubical expansion can be taken as three times this value.

The specific heat of mixtures appears to be primarily influenced by the specific heat of the aggregate, because it occurs in such large concentrations in asphaltic paving mixtures. An upper value of the order of 0.22 cal/gm°C (Btu/lb°F) in the range 0 to 25 C appears to be reasonable for mixtures with comparatively high asphalt contents.

Thermal conductivity data are somewhat limited. For well-compacted mixtures, however, this value appears to be of the order of 1 kg cal m/m²,°C,hr, or 0.7 Btu ft/ft²,°F,hr. From Eq. 22, it should be noted that thermal conductivity, k, is also influenced by the characteristics of the aggregate. It is probable that k-values for mixtures with higher void contents will be less than this value when they are in a dry state.

Barber and Monismith et al. have shown that temperature distributions at the pavement surface and within thicker asphaltic concrete layers can be estimated with a reasonable degree of confidence. From a knowledge of such temperatures and the rheologic behavior of asphaltic mixtures over a range in times of loading and temperatures (mixture stiffness), it has also been indicated that estimates of thermal stresses can be made. Although these estimates are by no means precise, they do give an indication to the engineer as to the probable range of temperatures where he can expect difficulties. In general, it appears that thermal stresses, by themselves, will not cause cracking at higher temperatures. However, in the lower temperature range below freezing it is possible that thermal stresses, whether by themselves or when added to the load stresses, may result in fracture of the mix. Thus, this situation should be considered by the design engineer where warranted.

DESIGN IMPLICATIONS

Based on the information presented in this chapter, it seems reasonable to conclude that thermal stresses can be calculated based on a knowledge of thermal and rheological properties of asphaltic concrete. It should be pointed out that these computations are actually quite involved and generally require some type of computer solution. However, the important factor to consider is that these stresses can be determined. The significance of these stresses has not as yet been completely evaluated. Monismith suggests, as indicated in Figure 54, that thermal stresses for all but the most severe conditions would not exceed the cold temperature fracture strength. He suggests, however, that the superposition of a wheel load during periods of low temperature could result in fracture. This will require further study, because wheel load stresses and strains during periods when the subgrade materials are frozen will be very small. It is possible that a critical condition occurs during periods when the subgrade is above 32 F and surfacing temperatures range from freezing to slightly above freezing. Under these conditions, the thermal stresses by themselves may not be excessive, possibly no greater than 200 psi. It is quite possible, however, that the additive thermal stress, coupled with loading due to traffic, will substantially decrease the fatigue service life of an asphaltic surfacing.

Consideration must also be given to the cyclic effect of thermal stresses on asphaltic surfacing. For example, computed thermal stresses in the asphaltic concrete during January on the WASHO Road Test ranged from approximately 150 psi in tension to 50 psi in compression in a 24-hr period. It is conceivable that this stress history could be causing microscopic areas of discontinuity between the asphalt matrix and the aggregate, which could lead to stress concentration and a reduction in the anticipated performance. This interaction between thermal stress and cumulative damage should be investigated further.

CHAPTER FIVE

FATIGUE OF ASPHALTIC CONCRETE

In this chapter the factors influencing the behavior of asphaltic surface mixes in repeated flexure are considered. Failures of paving mixtures resulting from repeated bending (fatigue) are typically represented by the type of cracking shown in Figure 55. This type of surface failure was recognized by Hveem and Carmany (*65*) in 1948 and referred to by Porter (*123*) in 1950. Reports of research conducted in England and Europe during the early 1950's have indicated a similar awareness of fatigue cracking as a factor in the performance of asphaltic surfacing.

In 1955, Hveem (*66*) presented definitive data relating pavement deflection and performance. On the basis of deflection measurements using permanently installed electronic gages, he was able to suggest maximum or limiting deflections for the satisfactory performance of asphaltic pavements. In this investigation performance was based on the amount of cracking present; therefore, it can be assumed that the limiting deflection was selected to preclude surface cracking. The limiting deflections suggested by Hveem are given in Table 20.

Although no definite number of load repetitions or service life was associated with these deflection values, it was implied that if deflections did not exceed these values during a reasonable service life, unlimited numbers of repetitions could be applied without flexural (fatigue) cracking.

To further demonstrate the validity of his argument, Hveem developed a laboratory fatigue testing device. This equipment was designed to subject a uniformly supported specimen to a constant deflection. Tests were made on field specimens obtained from the WASHO Road Test and from in-service pavements in California. Trends obtained from these tests are shown in Figure 56. On the basis of this information, it was concluded that laboratory fatigue tests could possibly be used to identify the relative fatigue performance of asphaltic concrete.

Since 1955, the highway engineering research literature has been replete with reports relating pavement deflection to pavement performance. (Of particular interest are Refs. *13, 14, 15, 21, 27, 32, 62, 66, 69, 106, 124.*) For the most part, these investigations have established that asphaltic pavements are subject to a loss of serviceability resulting from cumulative effect of tensile stresses less than the tensile strength of the surfacing. With few exceptions, no specific effort has been made to define the properties of the asphaltic surfacing which influence fatigue life, although some effort has been made in the laboratory to measure the effect of repetitive loadings on asphaltic concrete specimens.

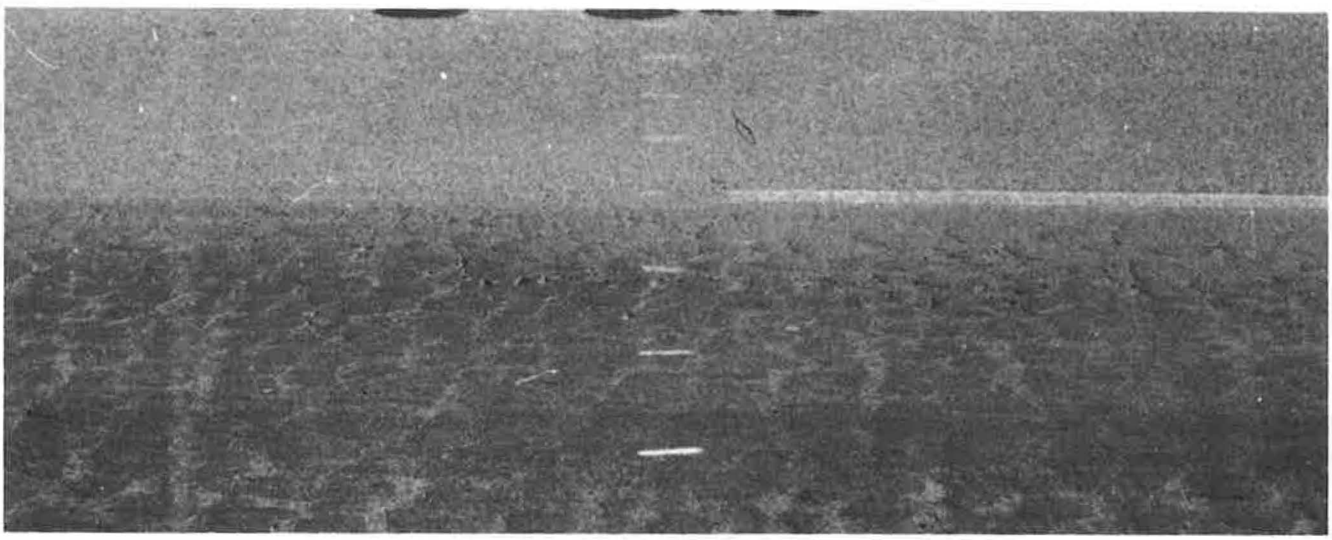


Figure 55. Typical fatigue-type cracking (after Hveem, 66).

This chapter reviews available information relative to research dealing with fatigue properties of asphaltic concrete and their influence on the performance of asphaltic surfacing.

DEFINITIONS AND GENERAL CONSIDERATIONS

To facilitate this review of fatigue and fatigue characteristics, it is necessary to define terms frequently used in the literature.

For a comprehensive treatise on terminology related to fatigue testing, *ASTM Spec. Tech. Publ. 91A* (1963) is recommended. Some of the definitions presented herein have been taken from this publication. Another complete compilation of fatigue terminology has been presented by Deacon (*25*).

FATIGUE (*ASTM STP 91A*). "The 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." An alternative definition is: "Phenomenon of fracture under repeated or fluctuating stress having a maximum value less than the tensile strength of the material."

SERVICE LIFE, N_s. "The accumulated number of load applications necessary to cause failure."

FRACTURE LIFE, N_f. "The accumulated number of load applications necessary to completely fracture a specimen." Quite often, as will be seen subsequently, $N_s = N_f$; however, this relationship should not be assumed to exist in all cases, because failure as defined by service life, N_s, may not correspond to complete fracture.

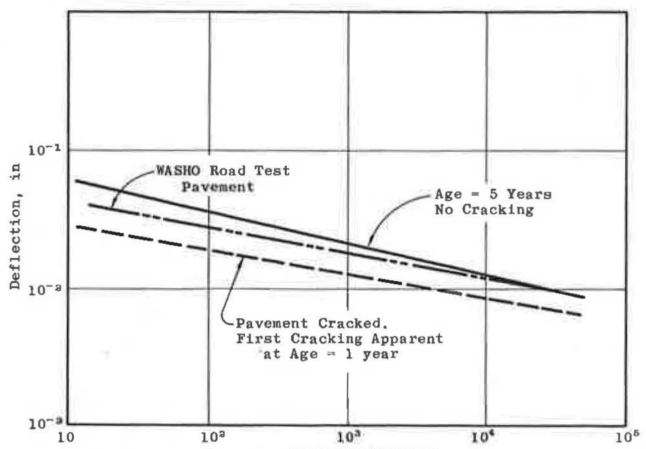


Figure 56. Relationship between repetitions and deflections for asphatic concrete at temperatures of 72-75 F, 2 × 2 × 10-in. beams (after Hveem, 66).

TABLE 20

SAFE MAXIMUM DEFLECTIONS [a]

PAVEMENT THICKNESS (IN.)	PAVEMENT TYPE	MAX. PERMISSIBLE DEFLECTION FOR DESIGN PURPOSES [b] (IN.)
8	Portland cement concrete	0.012
6	Cement-treated base (surfaced with bituminous pavement)	0.012
4	Asphaltic concrete	0.017
3	Plant mix on gravel base	0.020
2	Plant mix on gravel base	0.025
1	Road mix on gravel base	0.036
1½	Surface treatment	0.050

[a] After Hveem (*66*). [b] Tentative.

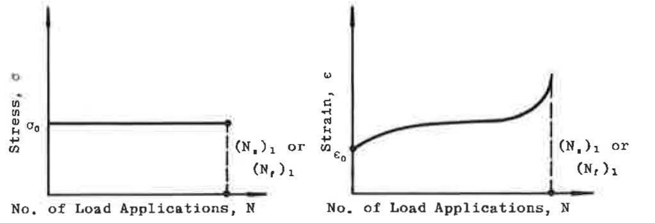


Figure 57. Idealized behavior in constant-stress fatigue test.

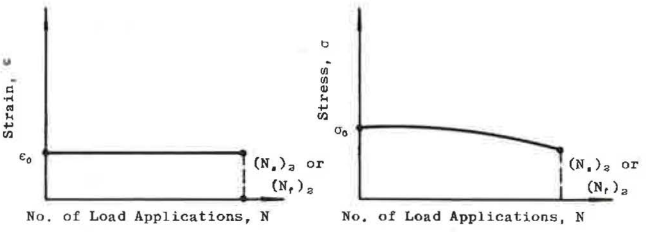


Figure 58. Idealized behavior in constant-strain fatigue test.

Laboratory Fatigue Tests

In the laboratory, fatigue behavior of materials such as asphaltic concrete has been determined in a number of different ways. Two of the most common are as follows:

1. Constant load or stress.
2. Constant deflection or strain.

For tests of the constant load or stress type, the load (stress) is maintained constant during each test. The relationships between stress and cycles to failure and between strain and cycles to failure as determined by constant stress tests are shown in Figure 57. It will be noted that strain (deformation) increases in this type of test until failure occurs; that is, until the service or fracture life is reached.

For tests of the constant strain (deformation) type, the strain is maintained constant until the service or fracture life is reached. Inasmuch as damage is usually progressive in some continuous manner, the stress (load) will decrease with increasing load repetitions, because the stiffness of the specimen will be decreased. Figure 58 shows the idealized behavior in this type of test.

Results of both types of tests can be plotted as shown in Figure 59, the familiar σ-N (stress vs cycles or repetitions of load) or ϵ-N (strain vs cycles or repetition of load) diagrams.

For the constant stress tests, if strain rather than stress is plotted versus cycles to failure, generally the initial strain is the value plotted; that is, the value of strain resulting when the specimen is undamaged. Also, in the constant strain test, if stress rather than strain is plotted, the initial stress is the value normally shown.

As would be expected, constant strain or constant stress type tests performed on replicate specimens may result in different fatigue lives, N_s or N_f. At low temperatures, differences in test results may be nonexistent or not significant. At higher temperatures, the difference in fatigue life between the two tests is biased, being greater in constant strain control, as shown by Figure 60. Some investigators believe that this difference between test methods is associated with the rate of crack propagation being faster in the constant stress test. Examination of Figures 57, 58, and 59 will help to explain this difference on the basis of the difference in energy input.

In the constant stress test the deformation increases

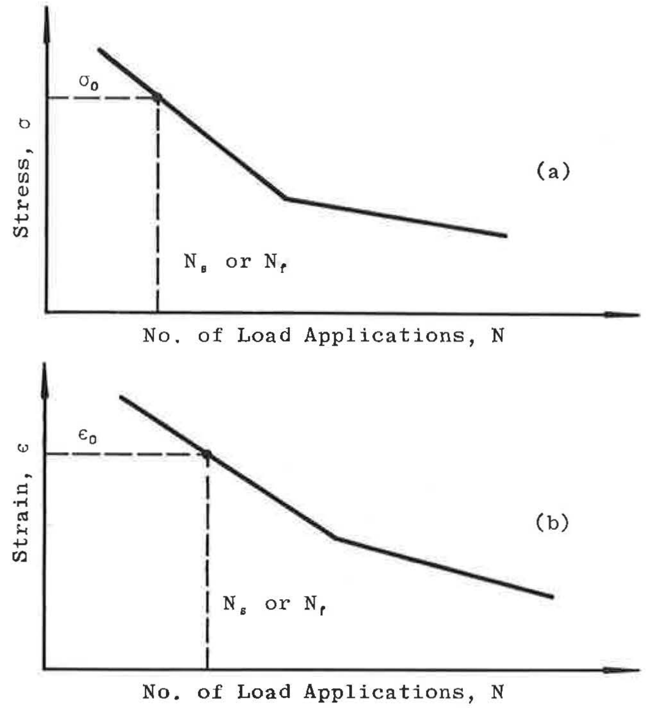


Figure 59. Stress and strain vs cycles to failure.

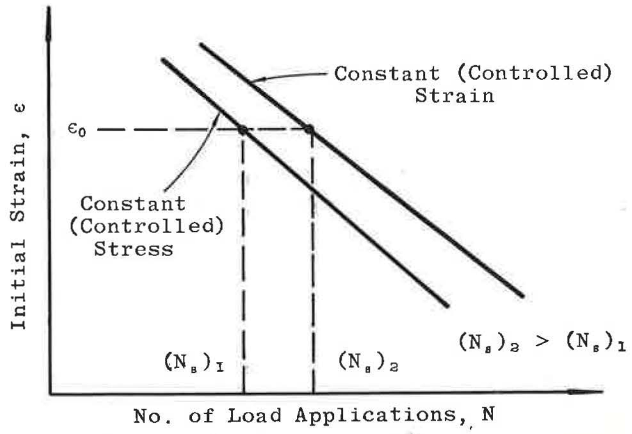


Figure 60. Comparison of controlled-stress and controlled-strain tests.

from its initial value, whereas in the constant strain test the stress is decreasing. Thus, the product of stress and strain increases in the constant stress test and decreases in the constant strain test. Because this product represents work, it can be seen that work (or energy) is expended more rapidly in the constant stress test, and a shorter life results.

Simple and Compound Loading

The preponderance of research reported in the literature has been based on fatigue studies with simple loading; that is, specimens are subjected to a series of unilevel stress or strain tests, and the corresponding cycles to failure are obtained. The level of stress or strain is usually selected to be representative of those to be encountered or anticipated in a pavement structure.

COMPOUND LOADING refers to a fatigue test performed with several levels of load. Deacon *(25)* discusses three compound-loading test procedures as follows:

1. SEQUENCE LOADING—applying different loads (usually, but not necessarily, two loads) in increasing or decreasing sequence; e.g., decreasing sequence would always start with the highest stress followed in order by the lower stresses.
2. REPEATED BLOCK LOADING—a defined sequence of block loadings applied repeatedly until failure occurs.
3. PSEUDO-RANDOM LOADING—a stipulated set of individual loadings randomly applied according to predetermined probability for each load.

Statistical Nature of Fatigue Data

In fatigue testing, the variation in the cycles to failure can be, and usually is, quite large. The ratio of the cycles to failure of essentially identical specimens subjected to a given stress level have been reported to be as high as 100 to 1. Thus, it must be recognized that the fatigue life of a material can only be realistically depicted as a distribution of values for a series of individual specimens. Because fatigue is a stochastic process, a sufficient number of specimens is usually tested so that estimates of the mean and median service lives and the service life which p percent of the specimens will equal or exceed can be obtained.

For a given stress or strain level, the distribution of service of fracture lives might take the form shown in Figure 61.

The MEDIAN FATIGUE LIFE is defined *(ASTM STP 91A)* as the middlemost of the observed fatigue life values, arranged in order of magnitude, of the individual specimens in a group tested under identical conditions. The MEAN FATIGUE LIFE is simply the average value of all the lives at a given stress level.

The FATIGUE LIFE FOR p PERCENT SURVIVAL *(ASTM STP 91A)* is the estimate of fatigue life that p percent of the population (survivals) would attain or exceed at a given stress level. For example, the median fatigue life is an estimate of the fatigue life for 50 percent survival. This is shown schematically in Figure 62. From this type of information, a series of curves of stress versus cycles to failure could be plotted, as shown in Figure 63. This curve is termed a P-σ-N diagram.

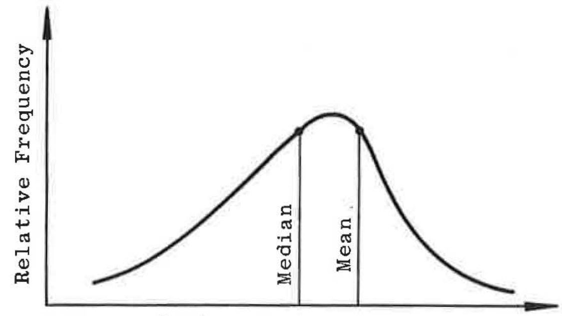


Figure 61. Mean and median fatigue life.

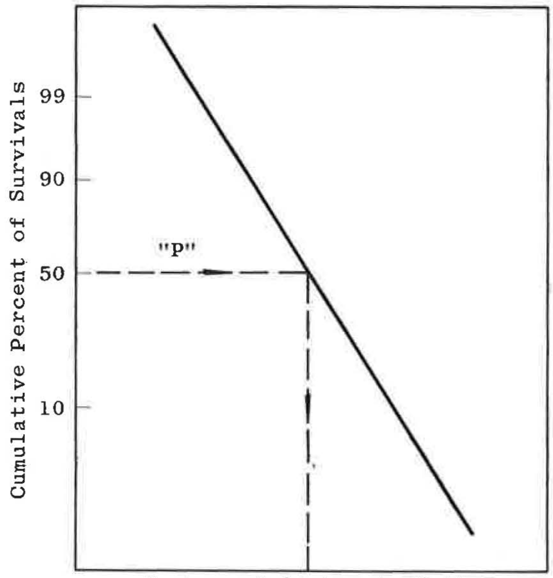


Figure 62. Service life based on percent survivals.

Generally, the scatter in fatigue lives is dependent on stress level, as shown in Figure 64. In simple loading tests, the pattern is usually such that as the stress increases the scatter decreases.

In general, the relation between service (fatigue) life and stress has the form:

$$N = a + \frac{b}{(\sigma\text{-}c)^d} \tag{23}$$

in which

N = number of cycles to failure (this may be mean or median service life, or service life at p percent survival);
σ = stress level; and
$a,b,c,d,$ = constants depending on conditions of testing.

Cumulative Damage

The term "cumulative damage" is normally used in reference to the analysis of compound loading tests. This technique is illustrated by Figures 65 and 66. Figure 65 is a "damage vs cycles" curve for a single stress level. Although "damage" is not specifically defined at this point, it can be thought of as some alteration in the physical properties due to cycles of load applications. By definition, at the service or fracture life, the damage level will be 1, or 100 percent.

There are many approaches which can be used to consider the influence of cumulative damage. A comparatively simple approach by Deacon (*25*) is discussed as an example.

Assume that a material has been subjected to a series of loads at two different stress levels and the "damage versus number of load repetitions" for each stress level is known, as shown in Figure 66. From these relationships it may be possible to predict the behavior in compound loading by summing damage increments as follows.

Assume that the specimen is subjected to N_1 repetitions of load condition 1; damage will be accumulated in the specimen as represented by the ordinate ΔD_1. If these repetitions are followed by N_2 repetitions of load condition 2, additional damage, ΔD_2, will result. This process can continue until

$$\sum_{i=1}^{i=m} \Delta D_i = 1 \tag{26}$$

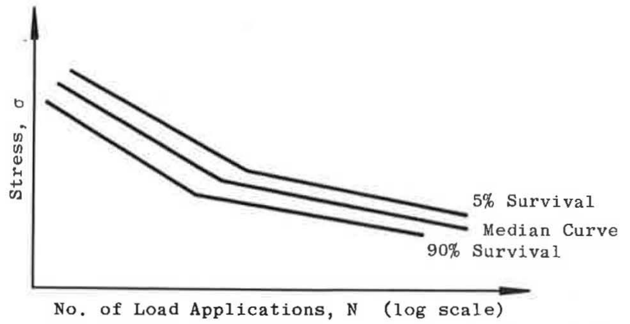


Figure 63. Estimated cycles at various levels of survival.

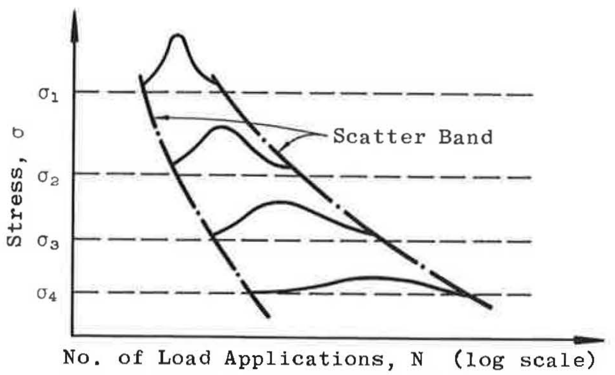


Figure 64. Variability in fatigue life as a function of stress level.

and the service life under compound loading becomes

$$N_s = \sum_{i=1}^{i=m} n_i \tag{27}$$

The process can apply to any number of stress levels so long as the damage versus number of load applications in simple loading is known for each stress level.

It should be noted that the method assumes that damage is irrecoverable and that stress history is not a factor.

By using this approach, the *linear summation of cycle ratios criterion* for cumulative damage can be developed. This is quite frequently referred to as the Miner criterion (Miner, 1945); however, it was developed earlier by Palmgren (1924) and Lawger (1937). According to Miner:

$$N_o = \frac{1}{\sum_{i=1}^{i=m} P_i/N_i} \tag{28}$$

in which

N_o = service life under compound loading;
N_i = service life in simple loading at load (stress) condition i; and
P_i = applied percentage of load (stress) condition i.

This equation expresses a relationship between mean fracture lives; unfortunately, it cannot be used to predict the distribution or standard deviation of the compound loading fracture life.

Deacon (*25*) has extended this method to develop a potentially more useful relationship by means of the following assumptions:

1. The logarithm of the fracture life for any particular specimen is linearly related to the logarithm of the stress level.

2. The slope of the relationship between the logarithm of the fracture life and the logarithm of the stress level is the same for all specimens.

This development is as follows:

Let

$$N_i = a\,\sigma^b \tag{29}$$

in which

N_i = simple loading service life for a particular specimen;
a = constant for that specimen; and
b = constant for all specimens represented by the slope noted in the foregoing assumption 2.

Using this equation, the service life for any specimen tested at any stress level, σ_i, is related to the simple loading service life, N_k, at standard stress, σ_k, by

$$N_i = \frac{(\sigma_i)}{(\sigma_k)} N_k \tag{30}$$

Applying Eq. 30 to the Miner criterion,

$$N_o = \frac{N_k}{\sum_{i=1}^{i=m} [P_i(\sigma_k/\sigma_i)^b]} \tag{31}$$

In this case the service lives shown are the mean service lives. The standard deviations of N_o can be found from

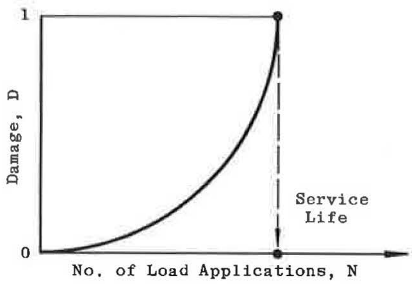


Figure 65. Damage vs number of stress applications.

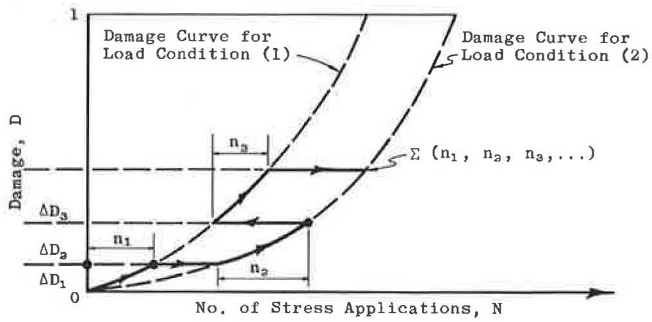


Figure 66. Procedure to summarize cumulative damage from simple load conditions.

$$S(N_o) = \frac{s(N_k)}{\sum_{i=1}^{i=m} [P_i(\sigma_k/\sigma_i)^b]} \tag{32}$$

Thus, a technique is available to consider the cumulative effects of different stress levels on single specimens tested in simple loading and some statistical measure of the variability in predicted results can also be obtained.

REVIEW OF LABORATORY AND FIELD STUDIES TO EVALUATE FATIGUE PROPERTIES OF ASPHALT CEMENT AND ASPHALTIC CONCRETE

For the most part, investigations of fatigue properties of asphalt and asphaltic concrete have been limited to simple loading, usually in constant stress or constant strain mode. Some variations of the test procedures between investigations will be subsequently noted. These differences make it difficult to make direct comparisons of test results. From the earliest studies of fatigue, researchers have attempted to use the rules of elasticity to evaluate test results. For example, stresses and strains are computed, from formulas of elasticity, as a means of defining the independent variable or as a means of associating service or fatigue life with a particular loading condition. To be able to do this, these investigators found it necessary to determine the modulus of elasticity of the materials used. As was discussed in Chapter Two, the term stiffness has been used in lieu of the modulus of elasticity by many investigators in an effort to recognize the rheological (time-temperature dependency) properties of asphaltic materials.

The ensuing section of this chapter describes the results of investigations dealing with the fatigue properties of asphalt and asphaltic concrete, including the influence of aggregate.

Factors Influencing Fatigue Properties of Asphalt Cement

Pell et al. (*118, 119*) have reported results of both constant stress and constant strain fatigue tests on asphalt cement. Figure 67 summarizes typical results of the constant stress tests. It is pertinent to note that the failure criterion for the constant stress type tests reported by Pell et al. was complete rupture of the specimen. It may be assumed that microscopic cracks develop within the specimen somewhat prior to this failure point. In Figure 67, the test data are described by a linear relationship between stress and cycles to failure plotted on log-log scales. It was noted by Pell that, although the data could be connected by lines which were generally parallel, a horizontal displacement was evident which appeared to be associated with temperature. In an effort to eliminate the temperature effect, Pell et al. converted stress to initial strain (Fig. 68). This was accomplished by determining the stiffness modulus of the asphalt cement using the method developed by van der Poel (see Chapter Two) and computing strain as a function of the method of loading.

The significant findings from this research effort with asphalt cement were as follows:

1. A linear relationship exists between stress or strain and cycles to failure when plotted on a log-log scale.
2. Plotting strain versus cycles to failure will tend to minimize differences in test results for cold temperatures (in these tests below 4 C).
3. Asphalt cement exhibits fatigue properties commonly associated with metals; specifically, failure under repetitive load applications less than the fracture strength of the material.

Figure 67. Fatigue results of bitumen specimens at various temperatures and speeds under constant bending stress (after Pell (118) by permission).

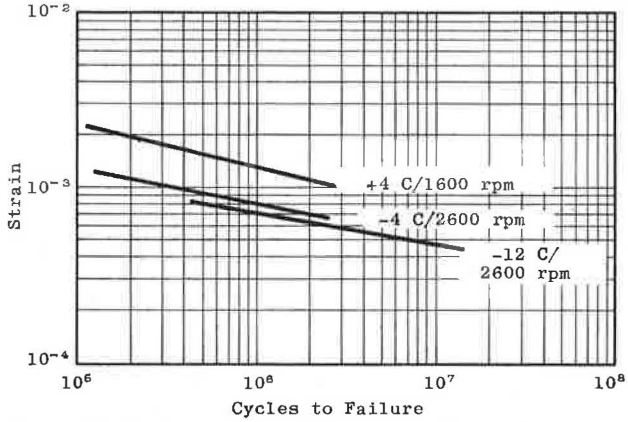


Figure 68. Fatigue results of bitumen specimens at various temperatures and speeds (after Pell (118) by permission).

To test the validity of the second of these conclusions, Pell (*118*) also performed fatigue tests on asphalt cement in constant strain. The failure criterion for these tests was modified somewhat from that used in constant stress due to the difference in loading history. Specifically, in the constant strain test continuous monitoring of the strain in the specimen is required and as damage begins the load must be reduced. Conceivably, complete fracture would not occur under this loading condition. In Pell's tests failure was detected by applying a conducting paint to the surface of the specimen; when a crack developed across the paint the circuit was broken, denoting failure. Figure 69 is typical of data obtained with asphalt cement in constant strain tests. The important results from these tests were as follows:

1. The relationship between strain and cycles to failure when plotted on a log-log scale is linear, confirming the results found in constant stress tests.

2. Strain tests conducted at low temperatures (< 4 C)

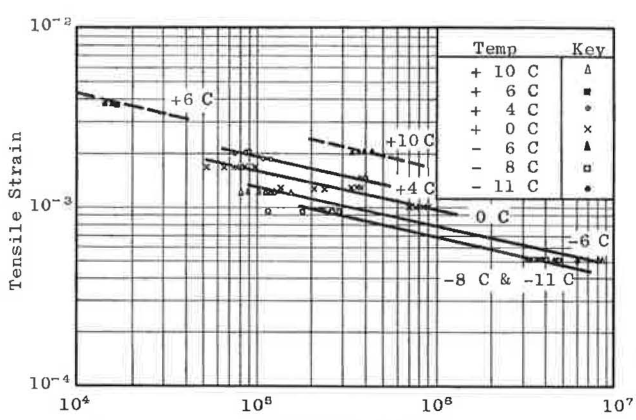


Figure 69. Fatigue results of bitumen specimens at various temperatures under constant torsional strain (after Pell (118) by permission).

tend to be independent of temperatures; at the warmer temperatures, however, the cycles to failure tend to increase with an increase in temperature.

3. Comparison of Figures 68 and 69 indicates the technique of converting from stress (in constant stress tests) to initial strain by means of a stiffness modulus will produce comparable results to tests performed in constant strain.

Pell et al. hypothesized from these and other data that the prime determinant of fatigue failure was tensile strain and that temperature effects could, therefore, be eliminated by plotting the logarithm of strain versus the logarithm of cycles to failure. Examination of Figure 69 seems to indicate that the use of tensile strain does not eliminate the temperature effect. However, Pell believes the apparent temperature-dependent differences in fatigue life can be reconciled on the basis of the failure criteria. Specifically, Pell (*118*) studied the failure surfaces of specimens tested in fatigue by means of photos. He concluded that all fatigue failures originate at some microscopic point of stress concentration in the specimen. For tests at lower temperature, this stress concentration results in an almost instantaneous fracture, whereas at warmer temperatures and lower stresses the crack progresses more slowly. The net result of this variation in the rate of crack propagation is to produce a bias in the test results, with greater cycles to failure for the warmer specimens.

It is pertinent to comment on the variability of the actual data shown in Figure 69. For tests conducted at 0 C, the cycles to failure for a strain of 1.8×10^{-3} range from about 5.2×10^{4} to 8.8×10^{4}. This variability increases as the strain decreases and poses a serious problem as to the number of tests required to define the fatigue properties.

It is significant that the research effort described herein tends to establish that asphalt does exhibit fatigue-like properties and that it would be logical to assume, therefore, that mixtures of asphalt and aggregate would tend to exhibit the same behavior.

Factors Influencing Fatigue Properties of Asphaltic Concrete

The purpose of this section is to review research, both laboratory and field, dealing with the fatigue properties of asphaltic concrete. Because of the complexity of the subject matter and the independent nature of the various research efforts, the material in this section is presented first as a discussion and description of various investigations, and finally as a summarization which attempts to develop a consensus relative to fatigue of asphaltic concrete.

Nijboer (*107*) was among the first to report on the results of repetitive tests of asphaltic concrete. As shown by Figure 70, Nijboer's fatigue tests on sandsheet mixes demonstrated fatigue properties similar to those reported for asphalt cement as indicated by the linear relationship between log of stress and the log of number of repetitions. On the basis of his data, Nijboer was able to suggest several important conclusions, as follows:

1. A limiting value of stiffness modulus for very short loading times and low temperatures was found to be ap-

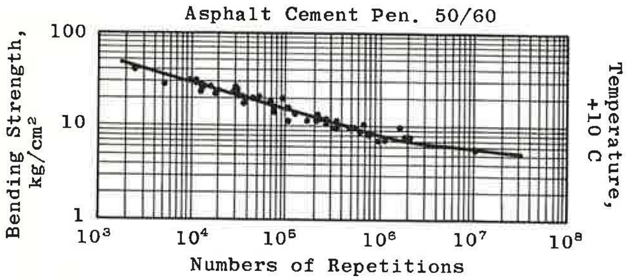


Figure 70. Fatigue test on sandsheet mixture (after Nijboer, 107).

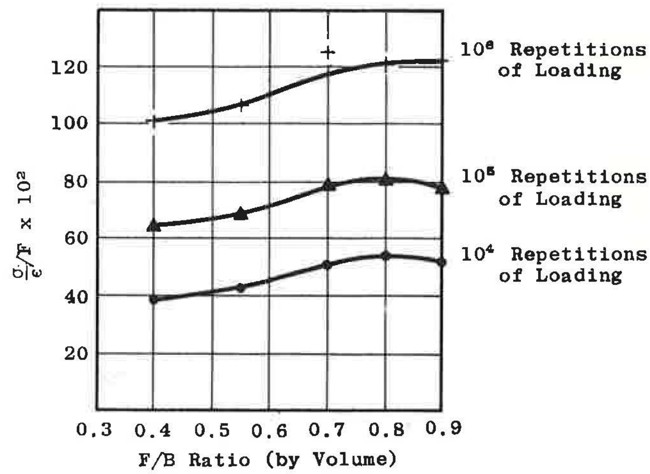


Figure 71. Relation between F/B *ratio (by volume) and* $\frac{\sigma}{\epsilon}$/F *for sandsheet mixtures at + 10 C; asphalt cement pen. 25 C = 50/60 (after Nijboer,* 107).

proximately 4,200,000 psi. It might be concluded that this stiffness would correspond to the modulus of elasticity of a purely elastic material composed of asphalt and aggregate.

2. For the sandsheet material, a maximum stress was achieved at between 2.5 and 3.5 percent deformation in compression and 1.5 and 2.0 percent in tension at higher temperatures, these values decreasing at lower temperatures.

3. The number of repetitions to failure is proportional to the bending stress, increasing with decreasing stress, as shown on Figure 70. The material thus shows the phenomenon of fatigue characteristic of elastic materials.

Nijboer postulated that, for most pavement construction, although the stiffness modulus would influence the strain in the asphaltic surfacing, the magnitude of the deflection and, therefore, the strain in the surfacing, would for the most part be controlled by the rigidity of the base, subbase, and subsoil. Accordingly, the stress in the surfacing would be proportional to the stiffness modulus of the mix. Thus, in keeping with the data shown in Figure 70, the fatigue life decreases as the ratio of stress to fracture strength, σ/F, increases. Experimentally, Nijboer found that the ratio of $\frac{\sigma}{\epsilon}\Big/F$ (stiffness modulus to fracture strength) increased with increasing values of the filler (minus No. 200 sieve)-bitumen ratio (F/B). This relationship is shown in Figure 71. He concluded that F/B ratio should be minimized in order to avoid crack formation in the surfacing. It could be concluded from this effort by Nijboer that mixes of lower stiffness modulus would be the preferred type.

Saal and Pell (*132*) and Pell et al. (*118, 119*) have conducted extensive laboratory research on sandsheet and mastic asphaltic mixes tested under simple loading in constant stress and constant strain. In their research, they have attempted to obtain fundamental information relative to the factors responsible for fatigue life. Studies included the evaluation of temperature, rate of loading, mix compositions, void content, surface finish of test specimen, rest periods, superimposed constant axial stress, and type of asphalt. In these investigations, rates of loading ranging between 800 and 3,000 cycles per minute, sinusoidally applied, were used in the constant stress determinations and 1,450 cycles per minute in the constant strain tests. These rates are somewhat higher than anticipated for highway loading. Although these higher rates may be expected to influence the order of magnitude of the effective stiffness modulus, it seems reasonable to assume that the ordering of fatigue life would be consistent with tests performed at lower rates.

Failure in the constant stress test was defined as rupture of the test specimen, whereas failure in the constant strain tests occurred when cracks on the surface of the specimen interrupted the electrical current flow through conducting strips painted on the specimen.

Figure 72 summarizes the results of constant stress tests in the range of − 13.5 to + 25 C. The general configuration of the data is similar to that of constant stress tests for asphalt cement. The tests indicate a temperature dependency with longer fatigue life at colder (stiffer asphalt) temperatures. In an effort to simplify the interpretation of the test results, stress was converted to strain by procedures similar to those described for asphalt cement. The stiffness modulus of the sheet asphalt was estimated in accordance with procedures of van der Poel described in Chapter Two. By converting stress to initial strain, Pell found the test results to be essentially independent of rate of loading and temperature, as shown in Figure 73.

Based on this type of analysis, it was concluded that constant strain tests would be more useful in evaluating fatigue properties. Therefore, tests were conducted in constant strain at temperatures from − 9.5 to + 40 C. The results of these tests are shown in Figure 74. Considering the statistical variations in the data, the results of tests at − 9.5, 0, and possibly + 15 C tend to substantiate the conclusions that tensile strain may be a primary determinant of fatigue failure. The data within these temperature limits also appear to be quantitatively similar to the results shown in Figure 73. In the case of tests at higher temperatures, such as + 30 and + 40 C, the results show a definite effect of temperature. Pell suggests that this difference is due to the difference in the test procedure and failure criterion.

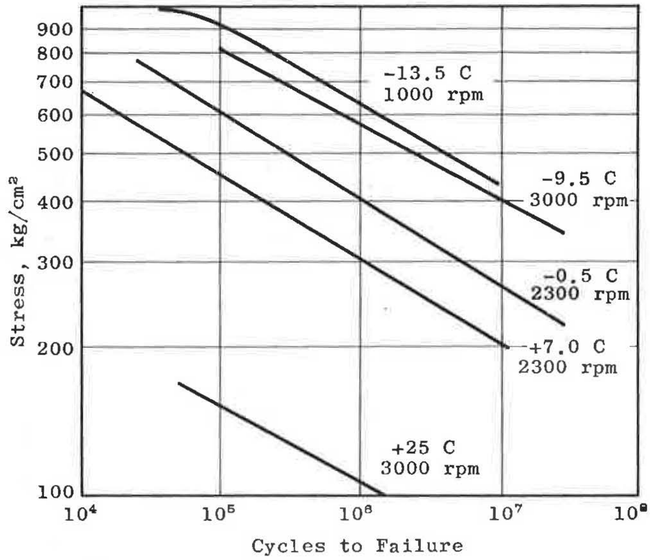


Figure 72. Fatigue results of sandsheet specimens at various temperatures and speeds under constant bending stress (after Saal and Pell, 132).

Temp °C	+25	+20	+7	-0.5	-0.5	-0.5	-9.5	-13.5
Speed RPM	3000	3000	2300	1000	2300	3000	3000	1000
Key	•	□	×	Ӿ	○	✦	+	△

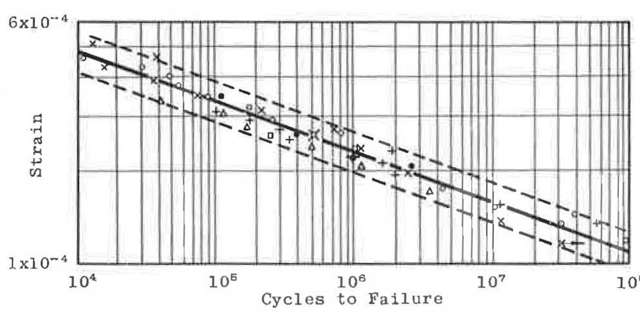


Figure 73. Fatigue results of sandsheet specimens at various temperatures and speeds (after Pell (118) by permission).

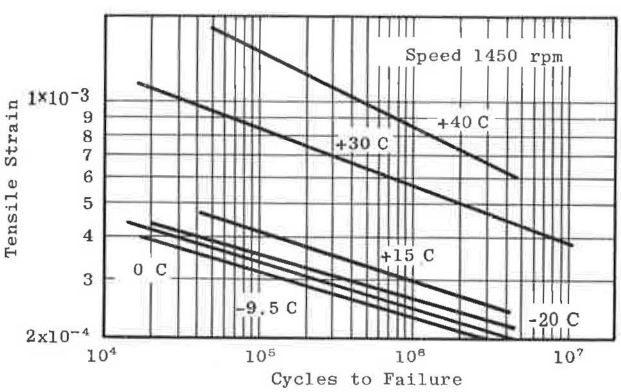


Figure 74. Fatigue results of sandsheet specimens at various temperatures under constant torsional strain (after Pell (118) by permission).

Specifically, he suggests that crack propagation is a function of the stress level at points of discontinuity; thus the warmer specimens (lower stiffness) would be subjected to less stress, therefore the rate of crack propagation would be slower, with a longer indicated fatigue life. This is similar to the conclusion reached for asphalt cement.

Pell, Saal, McCarthy, and Gardner have made important contributions to a better understanding of fatigue properties of sheet asphalt, and probably asphaltic concrete. The more important findings of their investigations may be summarized as follows:

1. The linear relationship between the log of stress or strain and the log of load repetitions found with asphalt cement was confirmed.

2. Tests performed in constant stress develop increasing fatigue life at low temperatures, whereas the opposite is the case for tests in constant strain.

3. It is hypothesized that tensile strain is the prime determinant of fatigue life and that tests in constant strain would be independent of temperature and rate-of-loading effects. Further, they suggest that fatigue life actually depends on the strain in the bitumen *per se*. A possible general equation for computing strain might be

$$\epsilon_B = \frac{\epsilon_m}{K_2 B_v} \tag{33}$$

in which

ϵ_B = strain in the bitumen;
ϵ_m = strain in the mix;
B_v = volume concentration of bitumen; and
K_2 = a constant depending on type of aggregate and volume concentration of aggregate.

4. By converting stress into initial strain, it was possible to develop an equation for estimating fatigue life independent of rate of loading or temperature, as follows:

$$n = 1.44 \times 10^{-16}(S/\sigma)^6 \text{ for } 10^4 < n < 10^8 \tag{34}$$

in which

n = cycles to rupture;
S = stiffness of sandsheet, psi;
σ = bending stress, psi; and
S/σ = $1/\epsilon$ (reciprocal of strain).

In a later publication (*120*) Pell suggested a more general equation for use in predicting fatigue life on the basis of initial strain, as follows:

$$N = K(1/\epsilon)^6 \tag{35}$$

in which K is a constant and ϵ is the initial strain.

5. The scatter band of fatigue life in constant stress was investigated, as shown in Figure 75, with a scatter band of ± 2 standard deviations covering a range of ± 200 percent.

6. The void content of the mix had an important effect on the fatigue life, as shown by Figure 76. Decreasing voids resulted in increased fatigue life.

Jimenez and Gallaway (*73, 74*) have reported results of fatigue tests on circular diaphragms made of sheet asphalt. The objectives of their research were to investigate the influence of such factors as specimen thickness, type

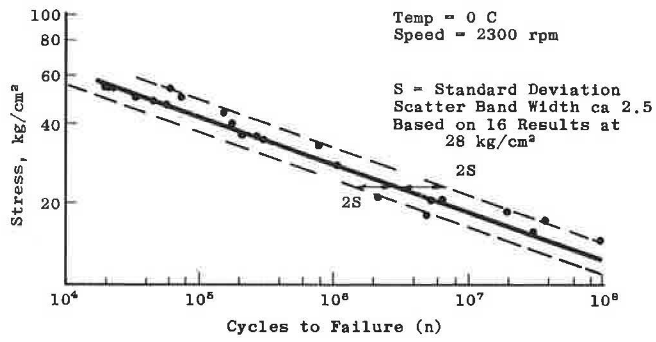


Figure 75. Fatigue results obtained with constant bending moment cantilever system (after Saal and Pell, 132).

of asphalt, and aggregate surface texture on the flexibility (magnitude of deflection) and fatigue life at 75 F. It is pertinent to indicate that the test method used by these investigators is neither strictly stress nor strain controlled. Both factors vary during the progress of the test. Based on the data and an examination of the loading system, it appears that the test could be classified as approaching the constant stress type. Figure 77 shows the deflection history during a single test and illustrates the failure criterion, which in this case is the deviation from a straight line when plotted to a log-log relationship. Tests on laboratory specimens were limited to sheet asphalts, although the authors also report testing field-produced mixes of asphaltic concrete.

In their interpretation of results, Jimenez and Gallaway used theoretical mathematical solutions developed by Morley for computing stresses and dynamic modulus of elasticity for the specific loading conditions. The equation used for radial (tensile) stress at the center of the diaphragm may be found in Ref. (*73*). For their work, Poisson's ratio was assumed to be 0.2.

Figure 78 shows test results on two mixes differing only as to aggregate surface texture. Two important relationships shown in this figure are (1) the longer fatigue life

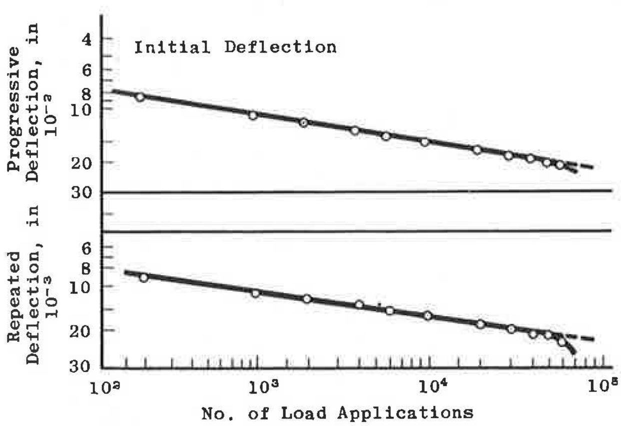


Figure 77. Load-disc deflection vs number of load applications (adapted after Jimenez and Gallaway (73) *by permission.*

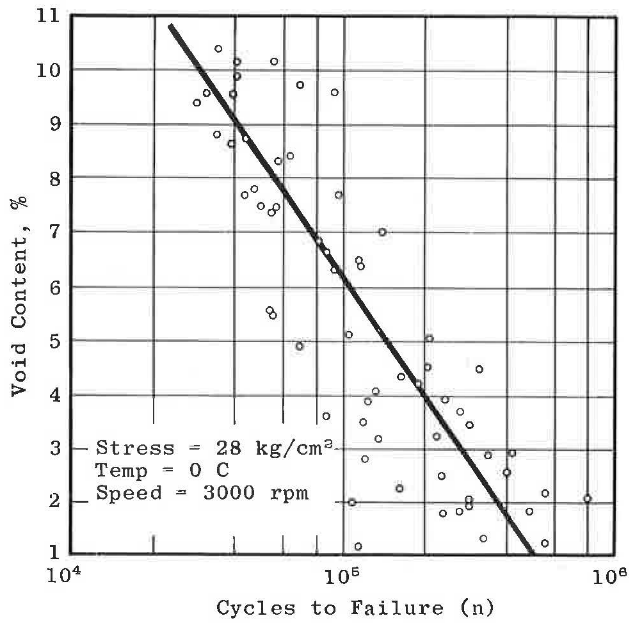


Figure 76. Effect of voids content on fatigue life (after Saal and Pell, 132).

of the rougher surface textured aggregate and (2) the optimum asphalt content indicated by each series of tests. The optimum asphalt content for each of the mixes shown in Figure 78, as determined by Texas Highway Department design procedures, was 6.5 percent. This would compare reasonably well with the information in Figure 78 for the

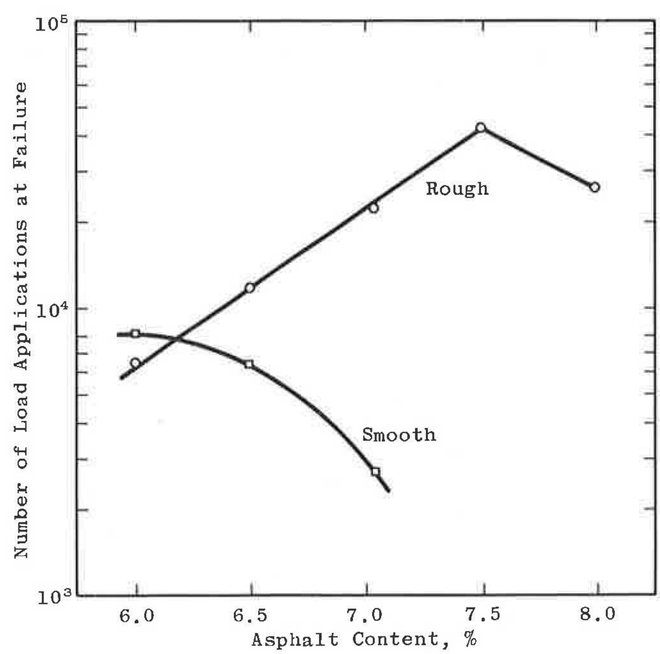


Figure 78. Asphalt content vs number of load applications at failure, standard (after Jimenez and Gallaway (73) *by permission).*

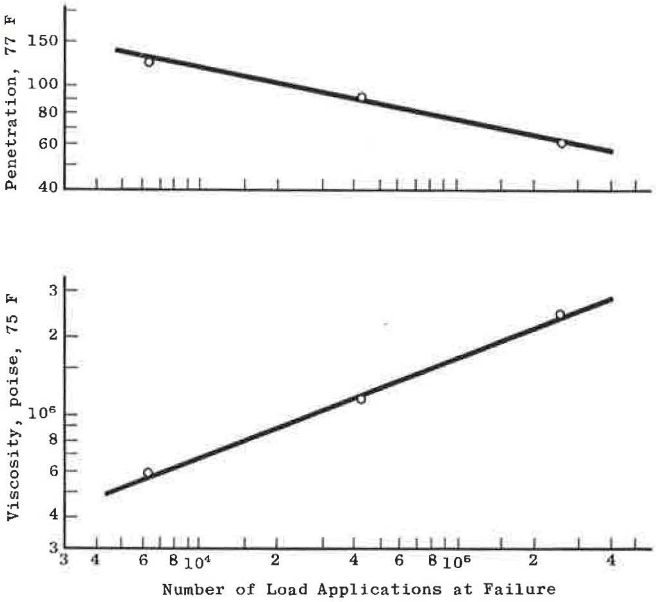


Figure 79. Asphalt viscosity and penetration vs number of load applications at failure (after Jimenez and Gallaway (73) *by permission).*

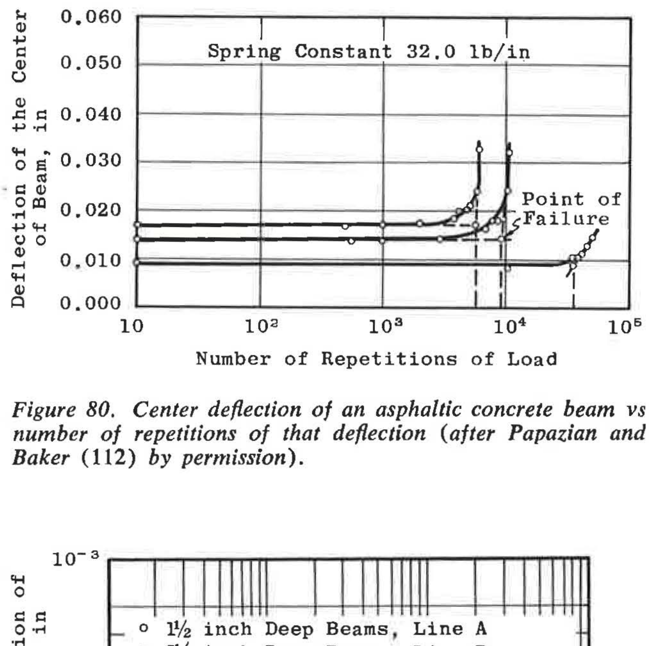


Figure 80. Center deflection of an asphaltic concrete beam vs number of repetitions of that deflection (after Papazian and Baker (112) *by permission).*

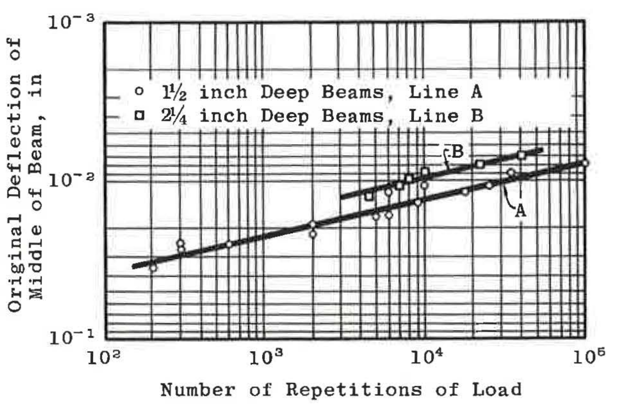


Figure 81. Relationship between center deflection and number of repetitions of that deflection to produce failure (after Papazian and Baker (112) *by permission).*

smooth aggregate, but not for the rough aggregate, the data indicating a longer fatigue life at 7.5 percent.

The pertinent findings from the work of Jimenez and Gallaway were as follows:

1. Sheet asphalt mixes made with aggregates of rough surface texture may produce a longer fatigue life than comparable mixes with smooth surface texture.
2. The optimum asphalt content, as normally determined by the Hveem stabilometer, provided a maximum fatigue life for smooth surface textured aggregate, but was deficient in asphalt for the rough surface textured aggregate.
3. The fatigue life of the sheet asphalt increased as the asphalt viscosity increased or the penetration decreased (Fig. 79).
4. Based on limited data, specimens of sheet asphalt cored from a pavement failed in repetitive loading at fewer cycles than laboratory mixes prepared with the same material. No detailed information is furnished as to void content of the two types of specimens.
5. The modulus of elasticity was affected by the specimen density, increasing with an increase in density.

Papazian and Baker (*112*) reported the results of an investigation relative to fatigue properties of asphaltic concrete. Their testing procedure consisted of placing a specimen on simple supports with a 9-in. span. A simply-supported steel leaf was placed transversely under the center of and at right angles to the specimen. In this way the beam was supported somewhat as a pavement by developing the interaction between the asphaltic surface and the supporting pavement layers. The stiffness of the steel leaf could be changed by changing the span length. The authors emphasized that the method was not intended to duplicate the performance of a pavement in the field, but to obtain a better understanding of the behavior of asphaltic concrete under repetitive deflections. Tests were made at 75 F and 105 load applications per minute (0.2-sec loading time) with specimens 1½ and 2¼ in. in depth. The failure criterion was similar to that used by Jimenez and Gallaway, as shown by Figure 80. The pertinent findings of their investigation were as follows:

1. The log of deflection versus the log of repetitions to failure can be plotted as a straight line (Fig. 81). As would be expected, for a given deflection the thinner beam exhibited a longer fatigue life. (This confirms Hveem's observations and recommendations between surface thickness and permissible deflection.) Within the limits of this investigation, doubling the thickness would require a 40 percent reduction in deflection in order to produce the same service life.
2. The number of repetitions of load to produce fatigue failure appears to be a function of stress. Figure 82 shows the linear relation between log of stress and log of the repetitions to failure computed from the data shown in Figure 81.

Papazian and Baker also presented data from a study by Chandrangu, who performed fatigue tests for a series of conditions representative of a changing subgrade modulus or subgrade condition. Typical results from his studies are shown in Figures 83 and 84. For these tests the modulus, E, of the asphaltic concrete and the moment of inertia, I, of the specimen were constant. In the figures, the right portion shows the relationship of load (abscissa) versus deflection or flexural stress for three subgrade conditions as designated by the K-value. In Figure 83 it can be noted that for each subgrade condition there is a unique relationship between deflection and service life. Figure 84 shows that when deflection is converted to stress the service life is independent of the subgrade properties, except as they influence stress. These tests tend to strengthen the importance of being able to convert deflection to stress as a criterion of performance. This would also imply that the use of deflection alone, for pavement evaluation, could be misleading or at least incomplete.

Monismith et al., at the University of California, have reported a series of fatigue investigations relative to the following variables:

1. Aggregate gradation.
2. Aggregate type.
3. Asphalt content.
4. Type and hardness of asphalt.
5. Temperature.
6. Rate of loading.
7. Strain reversal.
8. Methods of testing.
9. Compound and simple loading.

Monismith (*95*) describes the development of testing equipment used to study fatigue properties under constant loading on a Westergaard (coil spring base) type of foundation. This investigation used the modulus of rupture as a means of evaluation of the fatigue properties of asphaltic concrete. Specifically, a comparison of the modulus of rupture for the unflexed specimen to a companion specimen subjected to repetitive load applications was considered a measure of the damage due to flexing. This study was concerned with the effect of asphalt content and aggregate gradation on damage characteristics. Based on laboratory results of the type shown in Figures 85 and 86, the following conclusions were suggested:

1. For a given aggregate gradation, the retained modulus of rupture, after repetitive loading, was higher for the specimen with the higher asphalt content.
2. The dense-graded mixes (100 percent passing ½ in. and 9 percent passing No. 200) exhibited a higher strength retention, after repetitive loading, than the open-graded mixes (100 percent passing ⅜ in. and 0 percent passing No. 200).

Monismith (*96*) presented the results of fatigue tests aimed primarily at evaluating the effect of temperature on the fatigue damage to asphaltic paving mixtures. The investigation also included the factors of asphalt content and asphalt type. Monismith used constant load procedures on a spring foundation for these studies. He used two 85-100 penetration asphalts, one a conventional material representative of paving asphalts from the Pacific Coast and the other a catalytically air-blown asphalt. The temperature-viscosity properties of these asphalts are shown in Figure 87, together with those for a Newtonian asphalt. The pertinent findings of this investigation were as follows:

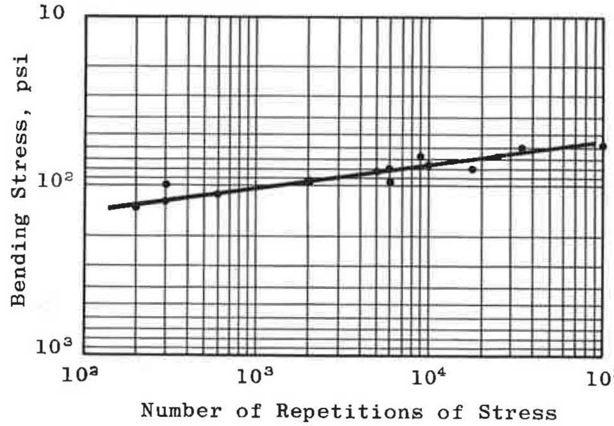


Figure 82. Relationship between bending stress and number of repetitions of that stress to produce failure (after Papazian and Baker (112) by permission).

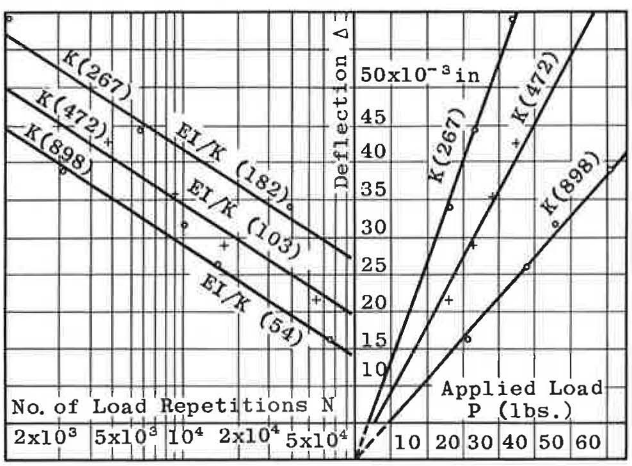


Figure 83. Relationship between load repetitions and specimen deflection (after Papazian and Baker (112) by permission).

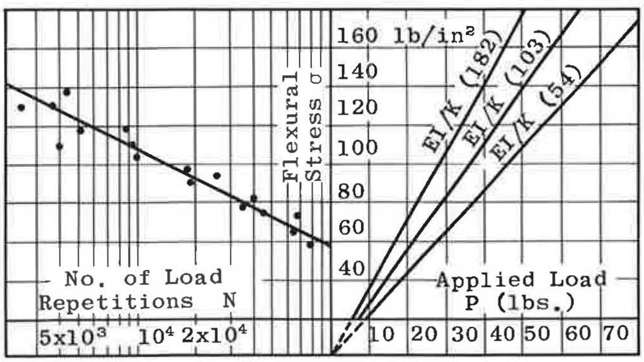


Figure 84. Relationship between load repetitions and calculated stress (after Papazian and Baker (112) by permission).

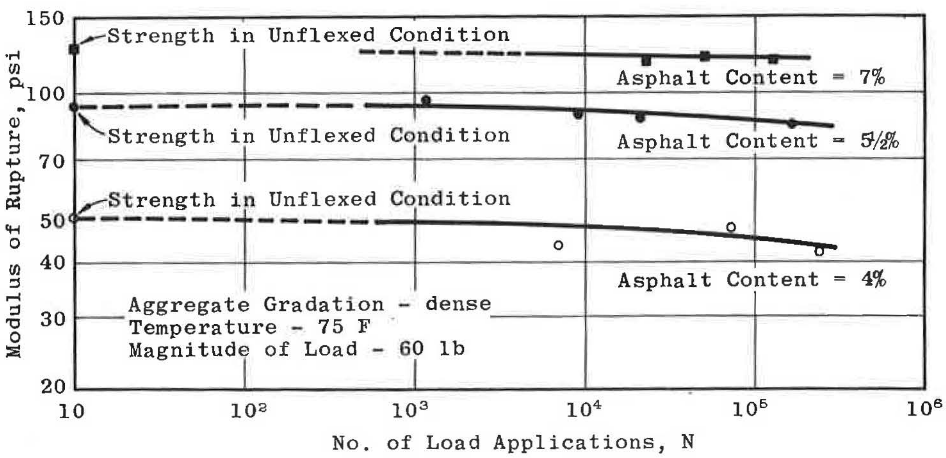


Figure 85. Effect of asphalt content on behavior of dense-graded mixtures subjected to repeated load applications (after Monismith (95) by permission).

1. As shown in Figures 88 and 89, with only one exception, the asphalts with the higher asphalt content exhibited higher strength retention in terms of their modulus of rupture.

2. There was little difference in the development of damage in the performance of the two asphalts at 40 F or 75 F.

At 75 F the airblown asphalts had a slightly higher retained modulus of rupture, the opposite being the case at 40 F. Figure 87 indicates this relationship is proportional to the asphalt viscosity at the test temperature.

Monismith (*96*) instrumented some of his specimens with strain gages in order to evaluate the strain induced by the 120-lb repetitive load. Two important findings resulted from this effort, as follows:

1. The measured strains were in general accordance with the theory of elasticity; specifically, compression in the upper fibers, tension in the lower fibers, and approaching zero strain at the centroid. This would tend to support the contention that the elastic theory can be applied to specimens of asphaltic concrete.

2. For a constant load, the strains in the outer fibers at 75 F were substantially greater than the corresponding strains in the specimens tested at 40 F.

Monismith, Secor, and Blackmer (*97*) in 1961 reported the results of laboratory studies to evaluate frequency of loading and stress reversal on fatigue life of asphaltic concrete.

In undertaking this investigation, Monismith and his

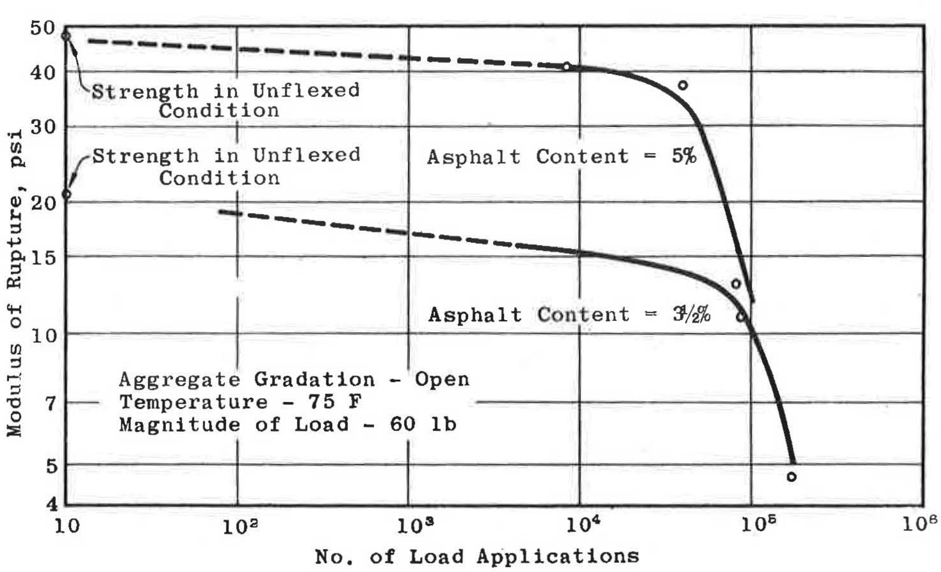


Figure 86. Effect of asphalt content on behavior of open-graded mixtures subjected to repeated load applications (after Monismith (95) by permission).

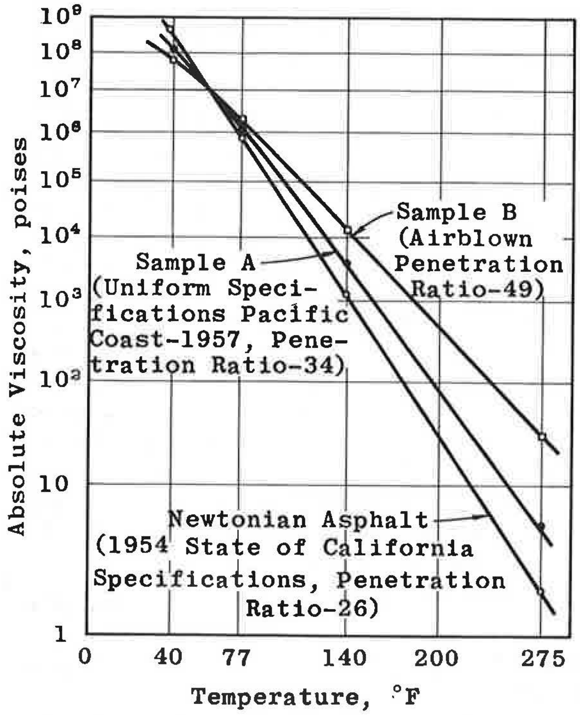


Figure 87. Temperature vs viscosity relationships for asphalt cements, with viscosity determined at shear rate of 5×10^{-2} sec^{-1} (after Monismith (96) by permission).

associates were attempting to reproduce, at least in a qualitative way, the potential influence of factors which could be anticipated to exist in an actual pavement situation. To represent a range of frequency variations (vehicle speeds), load was applied at 3, 15, and 30 applications per minute. To evaluate the stress reversal effects, three separate load deflection conditions were used, as follows:

1. Up 0.003 in. (⅓ sec), down 0.006 in. (⅓ sec), up 0.003 in. (⅓ sec), and rest (3 sec).
2. Up 0.003 in. (⅓ sec), down 0.003 in. (⅓ sec), up 0.003 in. (⅓ sec), and rest (3 sec).
3. Down 0.006 in. (⅓ sec), no upward deflection, and rest (3 sec).

For the studies of frequency effects and stress reversal, the method of evaluation was based on the relative damage induced by repetitive loading expressed as a ratio of the modulus of rupture after repetitive loading to the modulus of rupture of an unflexed specimen.

The two main conclusions from this investigation were as follows:

1. For the range of frequencies and loading used, there was no apparent effect of the rate of load application, as shown in Figure 90. Actually two asphalts were used in this phase of the research, a standard paving grade asphalt and an airblown asphalt; the results were the same for both asphalts.

2. Stress reversal appears to have little effect on the amount of cumulative damage to the asphaltic concrete.

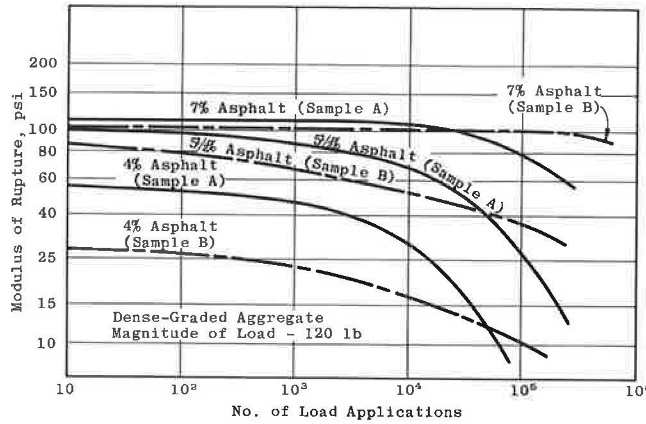


Figure 88. Comparison of behavior of specimens prepared with asphalt samples A and B in repeated flexure at 75 F (after Monismith (96) by permission).

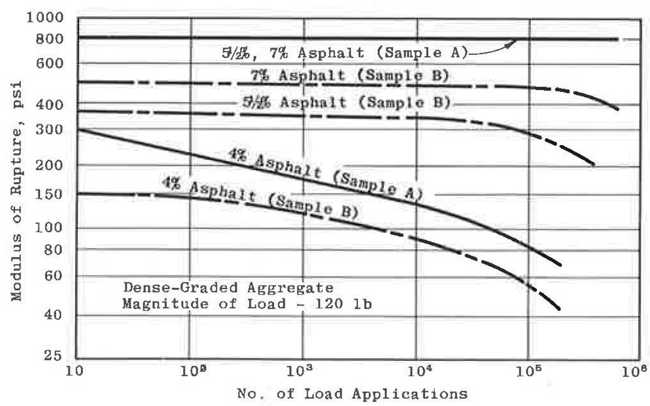


Figure 89. Comparison of behavior of specimens prepared with asphalt samples A and B in repeated flexure at 40 F (after Monismith (96) by permission).

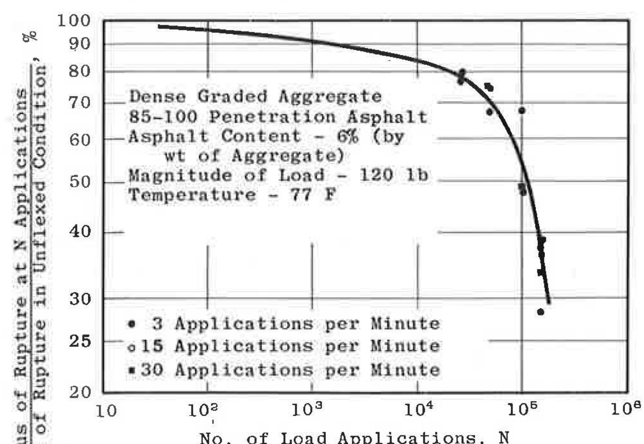
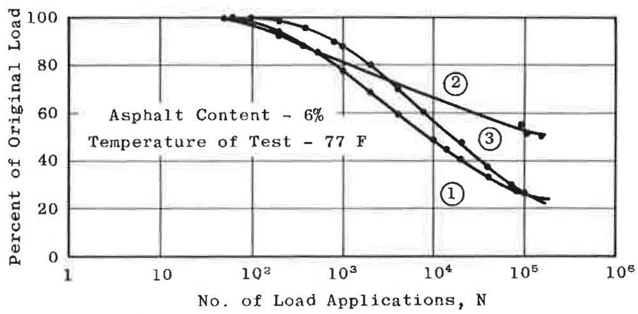


Figure 90. Effect of frequency of load application on behavior of dense-graded mixtures subjected to repeated flexure (after Monismith et al. (97) by permission).



Curve No.	Loading Cycle
①	.003" up (1/3 sec), .006" down (1/3 sec), .003" up (1/3 sec) and rest at zero deflection (3 sec), (3-6-3)
②	Same as Cycle 1, except only .003" down, (3-3-3)
③	.006" down movement (1/3 sec), (0-6-0)

Figure 91. Comparison of effect of various flexure patterns on strength reductions during stress-reversal studies (after Monismith et al. (97) by permission).

Typical results are shown in Figure 91. It should be noted that the magnitude of deflection, both up and down, was relatively small. Further investigations of this factor appear to be justified.

In the earlier discussions related to the work of Monismith, Secor, and Blackmer (*97*) it was noted that the inference had been made that for constant-load type testing the stiffer mix (higher stiffness modulus) resulted in less damage per load application. The opinion was expressed by two of the paper's discussers that this may not be the case for an actual highway because (a) the overall deflection, and consequently the strain, will be controlled not by the stiffness of the asphaltic surfacing but by the strength of the underlying substructure; and (b) actual service behavior indicates that the more flexible mixes are those with softer asphalts. The authors' (Monismith et al.) closure to these comments indicated their belief that the engineers' understanding of flexibility and fatigue may not be correct, or at least not completely understood, as regards the ability of a mix to withstand the repeated applications of loads. For example, in supporting their contention that the stiffness of the asphalt can significantly affect the pavement deflection, the authors referred to work of the British Road Research Laboratory. Figure 92 (*36*) shows that the stress measured at the top of the subgrade for the section containing the 50 penetration asphalt was approximately one-half of that measured in the section containing the 300 penetration asphalt. It would be expected that deflection and subgrade stress would be proportional. In their closure, the authors also pointed out that the characteristics of an asphalt manufactured to a low penetration value may be different from an asphalt which, through aging, has developed a low penetration. It was suggested that the composition of an aged asphalt might be such as to have poor fatigue properties.

It is pertinent to point out that although Monismith et al. in citing the work of the British Road Research Laboratory gave strong evidence of the load distribution potential of asphalt in asphaltic concrete, it does not necessarily follow that the fatigue life would be improved. This could only be established by measuring or computing the stress or strain in the surface layer and comparing this with the fatigue properties as determined from a fatigue test. It should also be noted in Figure 92 that the asphalt layer was approximately 5 in. thick. It would seem reasonable to expect that the asphalt consistency would be effective at this thickness level as compared to observations of asphaltic surface thicknesses of 2 to 3 in., where the effect is limited by the small proportion of surfacing relative to base and subbase.

Monismith (*99*) describes laboratory research to evaluate the effects of asphalt type and asphalt hardness. For this investigation, Monismith used a dense-graded crushed aggregate (100 percent passing ½-in. sieve and 7 percent passing No. 200 sieve) with five different asphalts. The asphalts were described as follows:

1. One 85-100 penetration asphalt cement (low penetration ratio).
2. One 85-100 penetration asphalt cement (used in asphaltic concrete pavements at AASHO Road Test).
3. Three asphalts from one source (medium penetration ratio):
   (a) 120-150 penetration.
   (b) 85-100 penetration.
   (c) 40-50 penetration.

These asphalts provided binders with three levels of penetration from three sources or refineries. The asphalt content used was approximately the same for all aggregates and asphalts, ranging from 5.8 to 5.9 percent. The asphalt content was selected according to California Division of Highways procedures and was considered representative of field mixes.

Monismith used constant strain tests in this study, monitoring the strains with a variable resistance bonded wire strain gage, nominally 1 in. in length. Fatigue properties were evaluated on the basis of retained modulus of rupture after repetitive loading when tested in simple flexure. The fatigue service life was defined as the number of load applications to cause a 10 percent reduction in the modulus of rupture.

Figure 93 summarizes results of constant strain tests with the three 85-100 penetration asphalts from three different sources. The identification of each asphalt corresponds to the foregoing listing. Two conclusions are indicated by these data, as follows:

1. The mixes with lower stiffness asphalts, as indicated by the higher temperature, exhibited the longer fatigue life.
2. Mixes made with asphalts of the same initial penetration from different sources may have different fatigue properties.

Examination of Figure 93 indicates that asphalt sample 2, used on the AASHO Road Test, has the superior fatigue properties. Monismith indicates that such a conclusion could be misleading. For example, because the tests were performed in constant strain, it would be pertinent to consider the load required to produce a specific strain in each specimen. From the laboratory data, the following loads were required to produce a strain of 350×10^{-6} in./in. at 40 F:

Asphalt Designation	Load (psi) Required for 350×10^{-6} in./in. Initial	Est. at Service Life [a]
Asphalt 1	490	380
Asphalt 3b	387	236
Asphalt 2	295	205

[a] Corresponding to 10 percent reduction in modulus of rupture.

Asphalt 1 could be considered superior on this basis, because more energy would be required to develop the same strain.

The conclusion that asphalt source could be a factor as regards fatigue properties could also be confounded by variations in asphalt properties not identified by initial penetration.

Monismith (*100*) reported on the influence of material properties and mix composition on fatigue behavior of asphaltic concrete with two types of aggregates and asphalts of three penetrations. The design asphalt content for each aggregate was determined by procedures of the California Division of Highways: 5.9 percent for the crushed granite and 4.5 percent for the uncrushed gravel. Evaluation of fatigue properties was made in the strain control mode, as previously described, using a spring foundation. A 10 percent loss in the modulus of rupture was again defined as failure under repetitive loading. The investigation was somewhat inconclusive as regards the influence of asphalt penetration on the fatigue. In general, the results of the strain-controlled tests indicated that the lower penetration asphalts produced mixes of higher stiffness modulus and lower fatigue life.

Figure 94 illustrates this finding for asphalts with penetration values of 42, 93, and 129. It will be noticed that the AASHO Road Test asphalts exhibited somewhat poorer fatigue service life properties at the lower strain values.

It was found that comparison of fatigue data based on stiffness modulus must be made at equal or nearly equal void content, because this factor can substantially influence the stiffness modulus. The lower the air voids, the higher the stiffness modulus. Figure 76, from work of Saal and

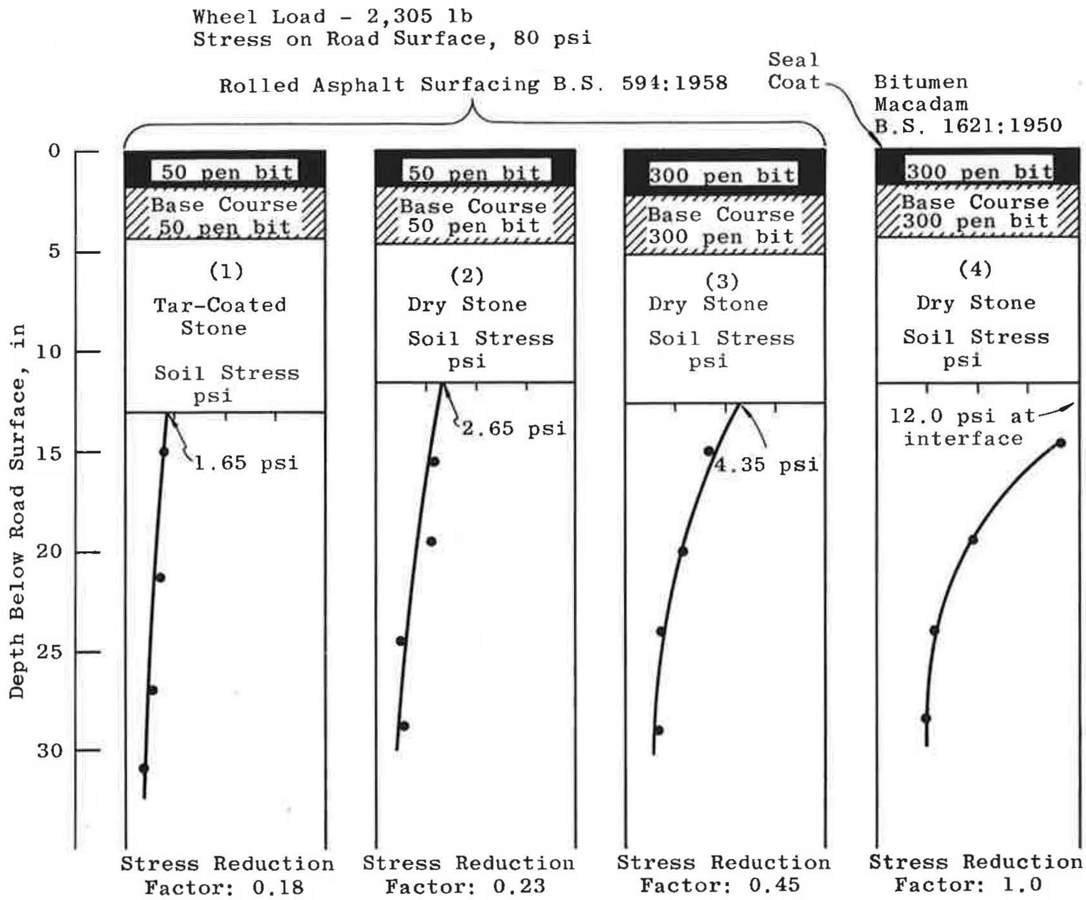


Figure 92. Loop road experiments: effect of type of road surface on soil stresses (after Fisher, Lee and Millard, 36).

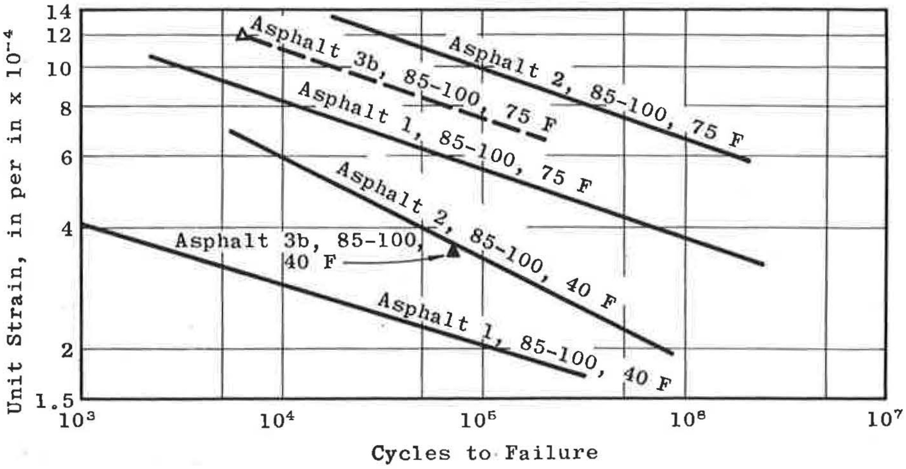


Figure 93. Summary of fatigue test data on specimens containing Watsonville aggregate and 85-100 penetration asphalts (after Monismith (99) by permission).

Pell, was used by the authors to indicate the general validity of the influence of void content on fatigue life of asphaltic mixtures. Considering the results obtained, Monismith concluded that density or void content was equally important with the penetration of the original asphalt in influencing the fatigue service life in constant strain testing.

Figure 95 graphically compares the somewhat limited data for crushed stone and the natural gravel in strain-controlled fatigue tests (*100*). On the basis of this information, the crushed stone appears to have the more desirable fatigue properties in constant strain. The actual significance of this difference, particularly at the lower temperature, may be questionable in view of the usual statistical variations in the laboratory data. However, this difference between crushed stone and natural gravel is somewhat more meaningful in view of the air voids in the two mixes (3.6 percent for the uncrushed gravel and 5.5 percent for the crushed granite). Thus, even with a higher amount of voids, the crushed rock produced a higher laboratory fatigue life.

There have been very few instances reported in the literature in which fatigue studies have been made both in the laboratory and with specimens taken from in-service pavement. One such investigation was reported in connection with the Shell Avenue Test Road (*39*) in Contra Costa County, Calif. The constant strain mode of testing was employed, and the failure criterion was based on a 10 percent reduction in modulus of rupture. The results of laboratory tests are summarized in Figure 96. Two grades of asphalt were used on the project—40-50 penetration and 85-100 penetration from the same supplier.

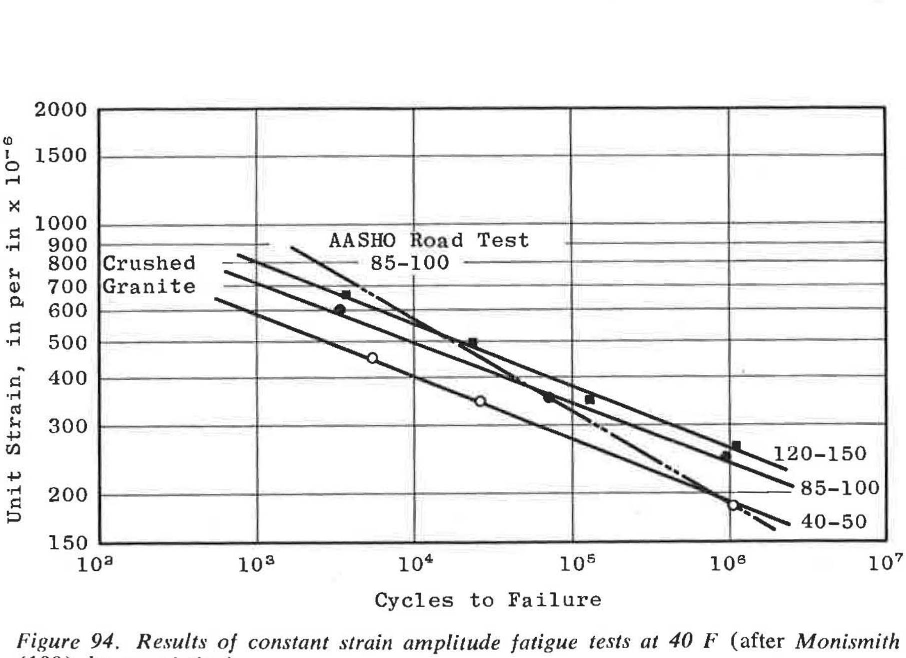


Figure 94. Results of constant strain amplitude fatigue tests at 40 F (after *Monismith* (100) *by permission*).

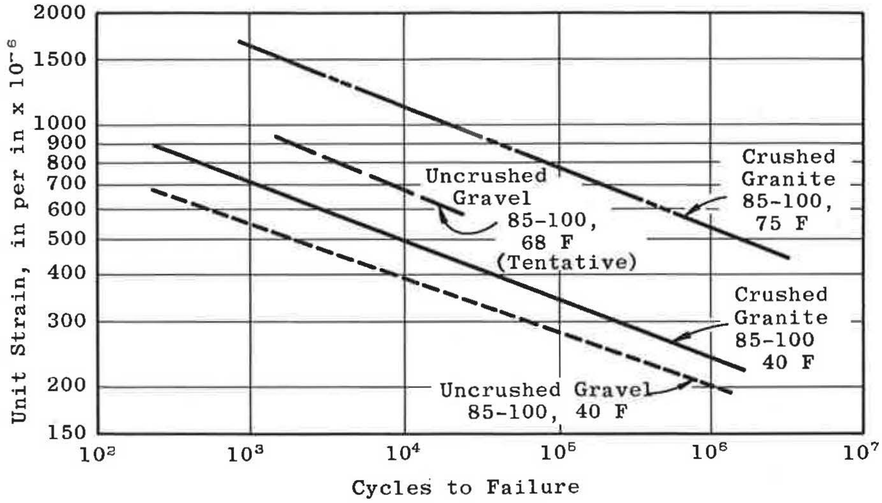


Figure 95. Influence of temperature and aggregate type on results of constant strain amplitude fatigue tests using medium penetration ratio asphalt (after Monismith (100) by permission).

From Figure 96, the following conclusions are suggested:

1. For asphalts from the same source, initial penetration may have no significant effect on fatigue in the constant strain mode of testing.
2. Temperature appears to have an effect on fatigue life, with the greatest number of cycles being associated with the higher temperature.
3. Good correlation is obtained between laboratory specimens and field specimens tested immediately after construction.
4. Field specimens tested 18 months after construction indicated a reduction in cycles to failure, a result which has probably been influenced by (a) fatigue damage due to traffic and (b) aging of the asphalt.

It is emphasized that these conclusions must be considered in the light that they are based on constant strain fatigue tests. Particularly, the conclusion relative to the initial penetration cannot be strictly interpreted until a determination is made as to the effect of asphalt penetration on strain in the actual pavement.

Ref. (*39*) also includes a tabulation of longitudinal and transverse strain in both the top and bottom surfaces of the 3-in. asphaltic concrete overlay placed on the Shell Avenue Test Road.

On the basis of measurements made on five different dates, Table 21 summarizes the strain measurements in the sections using 40-50 and 85-100 penetration asphalts.

Considerable care must be taken in drawing conclusions from this information. However, it should be possible to conclude that, on the average, the strain measurements in the sections with 40-50 penetration asphalt are substantially

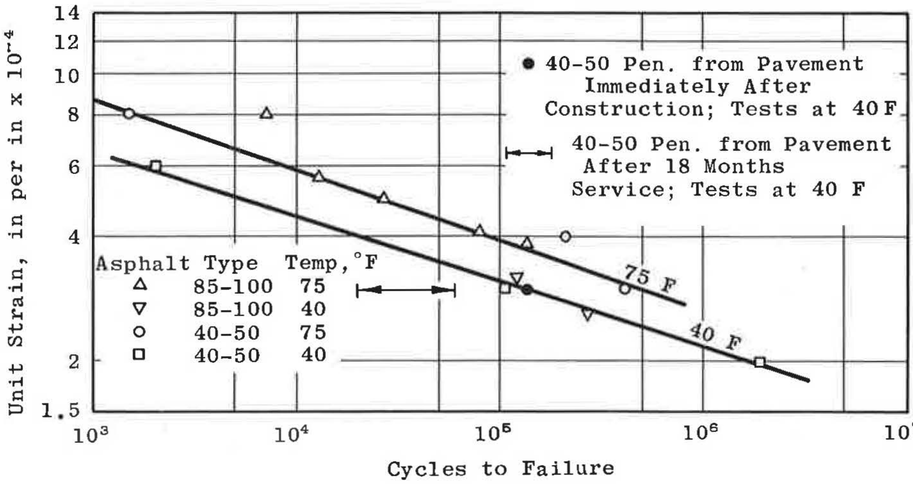


Figure 96. Results of constant strain amplitude tests on materials used in Shell Avenue Test Road, Contra Costa County (after Garrison, 39).

lower than those in the section with 85-100 penetration asphalt. Referring to Figure 96, it would be expected that the fatigue life with the lower pentration asphalt binder would be superior for the two grades used.

Unfortunately, within the 40-50 penetration or 85-100 penetration asphaltic concrete no observable cracking has developed after three years of performance; therefore, the superior performance of the 40-50 asphalt cannot as yet be supported by field observations.

Deacon (*25*) describes a research effort to develop a better understanding of the fatigue behavior of asphaltic surfacing material, with emphasis on laboratory test procedures and with interpretations to established fatigue theories. The specific variables included in his study pertinent to this report were as follows:

1. Review of laboratory techniques to evaluate fatigue behavior of metallic test specimens.
2. Develop laboratory equipment for applying controlled, repetitive load sequences to asphaltic concrete specimens.
3. Evaluate fatigue behavior in simple and compound loading.

He also deals with the statistical characteristics of the fatigue service life when tested in constant stress.

The equipment developed was a symmetrical, two-point load system with uniaxial bending stresses on a simply supported asphaltic concrete beam. The specimen is thus subjected to a constant moment throughout the center portion for a predetermined time, after which it is forced to return to its original undeflected position. More detailed information describing the test equipment are given elsewhere (*25, 26*). Deacon used an 85-100 penetration asphalt cement with a ⅜-in. maximum size aggregate for his investigations. He performed fatigue tests with both simple and compound loading with stresses ranging from 78.5 to 507 psi, depending on the type of loading, temperature, and objective of the test series. For his compound loading studies he chose increasing sequence, decreasing sequence, repeated-block, and pseudo-random types of loading. For evaluating performance in constant stress fatigue testing, he defined failure as the number of cycles required for the asphaltic concrete beam to rupture.

Results of simple loading tests generally confirmed, with probably better definition, the findings of other investigators (Saal, Pell, Papazian, Baker, Jimenez, Monismith, etc.) He reports, for example:

1. The functions, log N_f (mean cycles to failure) to the log σ (stress) or log ϵ (strain), may be approximated by linear relationships.
2. The initial stiffness modulus can be used to explain some of the variability in fatigue behavior, and a plot of log N_f to log ϵ exhibits less variability than log σ-type plots.
3. Two properties of asphaltic concrete test specimens that are altered or exhibit damage by flexural-type fatigue loading are the modulus of rupture and the stiffness modulus, both properties decreasing with repetitions of loading.
4. The internal temperature of asphaltic concrete specimens does not increase measurably (beyond approximately 1 F) during flexure testing at loading frequencies of 100 applications per minute or less.
5. The frequency of loading has a very significant effect on fatigue behavior, between 30 and 100 applications per minute, a reduced fatigue life being associated with the higher rate of loading.
6. Load duration affects the fatigue behavior, a longer duration resulting in a decreased fatigue life.
7. Specimens having higher specific gravities tend to have longer fatigue life. (This is for the case of one aggregate and asphalt and compaction procedure and is entirely consistent with results of other investigators.)
8. The most general damage determinant is the initial maximum principal tensile strain in the asphalt. (Deacon

TABLE 21

STRAIN IN ASPHALTIC SURFACING, SHELL AVENUE TEST ROAD [a]

			TENSILE STRAIN (IN./IN. × 10^{-6})[b]			
			BOTTOM		TOP	
DATE	ASPHALT PEN.	AC TEMP. (°F)	TRANSV.	LONG.	TRANSV.	LONG.
26 Oct. 61	40–50	76–78	26	75	75	50
26 Dec. 61	40–50	60–66	20	20	15	25
18 Apr. 62	40–50	80–86	15	80	85	45
30 Aug. 62	40–50	76–84	70	75	70	50
12 Apr. 63	40–50	80–90	35	115	40	60
Avg.			*33*	*73*	*57*	*46*
25 Oct. 61	85–100	79–80	170	280	115	100
26 Dec. 61	85–100	62	60	125	525	150
19 Apr. 62	85–100	94	575	500	150	240
30 Aug. 62	85–100	114–120	365	420	450	150
13 Apr. 63	85–100	70–84	350	345	115	175
Avg.			*304*	*334*	*271*	*163*

[a] Adapted after Garrison (*39*).
[b] Obtained with 15,000-lb single-axle load.

presents no data to substantiate this conclusion and depends to some extent on Pell's (*118*) analysis.)

9. Much of the uncontrolled variability in the fatigue behavior of the asphaltic concrete test specimens is dependent on the heterogeneity of the specimens.

Deacon discusses two properties of asphaltic concrete which can be associated with fatigue damage; namely, stiffness modulus and the modulus of rupture. The need to be able to evaluate damage is important if the concept of cumulative damage is to be used to predict fatigue life in compound or random loading. Damage as a function of the reduction in modulus of rupture with stress history was described by Monismith. Deacon, however, suggests measuring damage in terms of the reduction of the stiffness modulus, which can be measured in nondestructive tests. He investigated several methods for determining changes in the stiffness modulus, including measurements of resonant frequency. He recommended using the simple-beam flexure device used to measure the cycles to failure in fatigue tests. Measurements of deflection, as shown in Figure 97, illustrate how damage occurred in his test equipment.

Recognizing that the mean stiffness modulus is dependent on the stress level, Deacon suggests expressing the change as a percentage of the initial modulus; thus, it would range from 0 at a cycle ratio of 0 to 100 percent at a cycle ratio of 1. Figure 98 shows this procedure for four stress levels used in his investigation. On the basis of his findings, Deacon has suggested that damage could be defined by

$$D^i(R) = \left(\frac{100}{n^i}\right)\sum_j\left[\frac{(E^i_{oj}-E_{Rj})}{E^i_{oj}}\right] \tag{36}$$

in which

$D^i(R)$ = the damage as a function of cycle ratio, load condition i;

E^i_{oj} = the initial stiffness modulus for load condition i, specimen j;

E^i_{Rj} = the stiffness modulus for load condition i at a cycles ratio of R, specimen j; and

n^i = the number of specimens tested at load condition i.

Deacon investigated three types of compound loading, as follows:

1. Sequence loading.
2. Random loading by (a) repeated blocks of predesignated loads, and (b) pseudo-random loading.

From these investigations, he proposed an analytical method of describing fatigue behavior in compound loading.

The test conditions for the compound loading studies were 75 F, 100 applications per minute, 0.10-sec load duration, and constant stress loading.

The effect of sequence of loading was investigated by means of two-level increasing and decreasing sequence tests. Stress levels of 98.5 and 128.5 psi were employed. To illustrate the procedure, the decreasing sequence tests were performed by first applying all of the higher stress

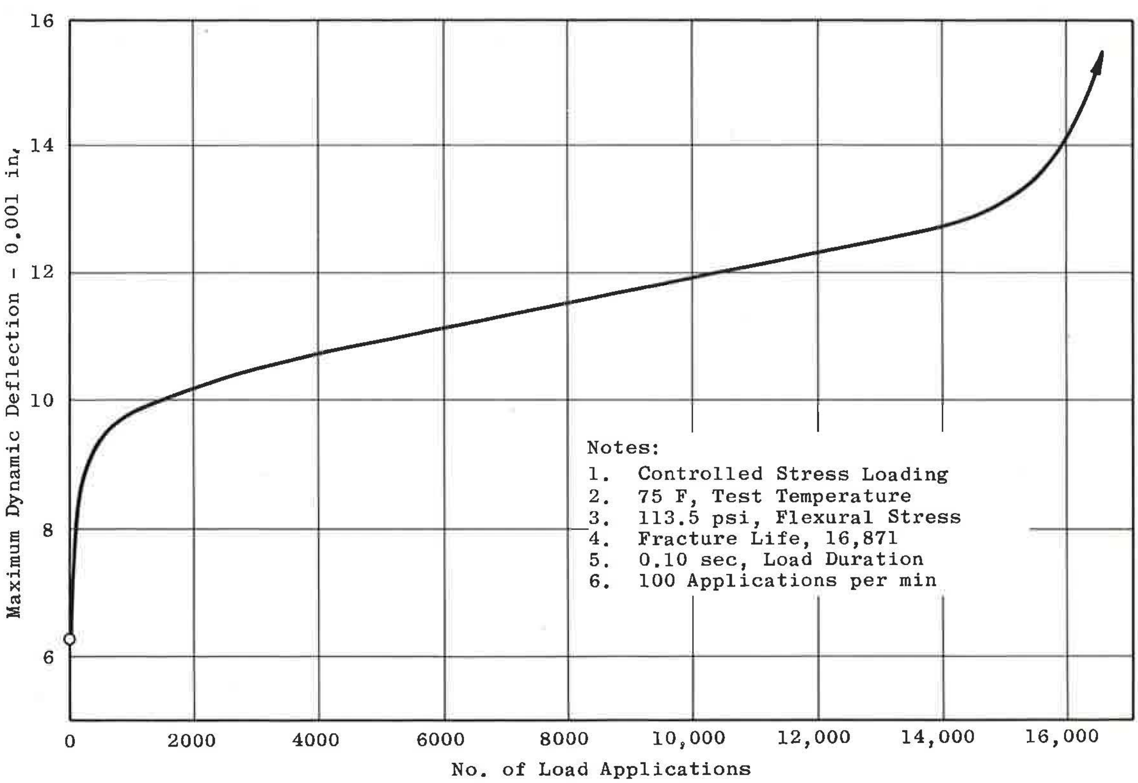


Figure 97. Dynamic deflection as a function of number of load applications for a typical specimen subjected to simple loading (after Deacon, 25).

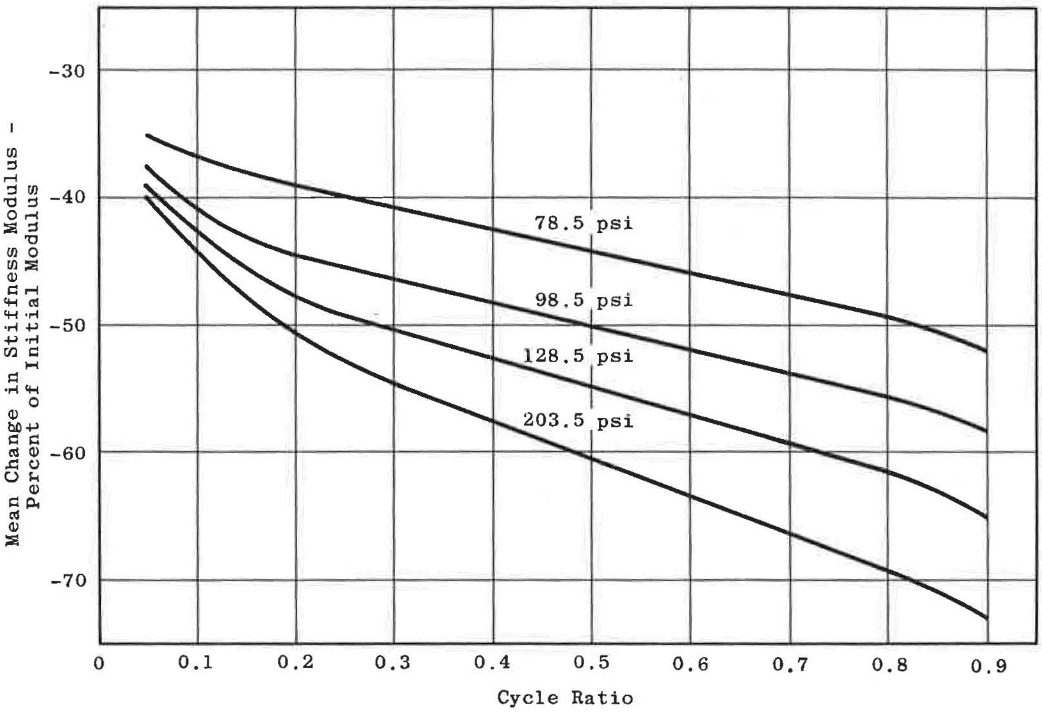


Figure 98. Alteration of mean stiffness modulus with cycle ratio, simple loading (after Deacon, 25).

level load applications, followed by the lower stress level. The primary independent variable was the total number of load applications required to determine the service life. Although he observed some minor differences in the mean fracture life, as a function of increasing or decreasing sequence, it was concluded that serious errors would not be introduced as a result of variations in the load sequence.

To more nearly represent in-service loadings, it would be desirable to perform fatigue tests with random loading. Unfortunately, random loading poses some major problems; for example, more complicated and expensive test equipment and a more involved or complex analysis of test data. Deacon attempted to evaluate the feasibility of using repeated-block or pseudo-random loading as a substitute for random loading. Deacon (*25*) described repeated-block loading as follows: "A repeated-block test is a test in which a block of load applications is applied repeatedly until failure occurs, each subsequent block being identical to that which precedes it." Random loading was also simulated by a pseudo-random type loading in which limitations are placed on the number of choices for the loading and the probability for each load; however, the sequence of loading is based on a random selection. Repeated-block loads and pseudo-random loads were the same as regards magnitude. Table 22 summarizes the comparison of the pseudo-random and repeated-block loading test results.

The pseudo-random tests show longer fatigue life in Series A and B, shorter in Series C. It is significant to note that the fracture life (fatigue life) appears to correlate with the initial stiffness modulus better than with the type of loading. Based on data of this type, together with considerations as to the variability in the test data, Deacon concluded that the mean fracture life by either mode was the same.

The final objective of the compound-loading test was to explore various mathematical models which could be used to predict fatigue life. Deacon was able to develop equations for both the mean fracture life and the standard deviation of the mean fracture life, as follows:

$$Y_2 = \frac{b}{\Sigma_i P_i \, (S_k/S_i)^b} \tag{37}$$

in which

Y_2 = predicted mean fracture life;
k = average fracture life at standard stress level, S^k, simple loading;
P_i = applied percentage of load condition i;
S_k = any standard stress; and
b = a constant equal to the slope of the linear log N_f-log S for simple loading.

and

$$S(N_f) = s(N_f^k)\left[\frac{1}{\Sigma}(S^k/S_i)^c\right] \tag{38}$$

in which

$S(N_f)$ = the standard deviation of the compound-loading fracture life;
$s(N_f^k)$ = the standard deviation of the simple-loading fracture life at standard stress level, S^k; and
c = a constant equal to the slope of the linear log $s(N_f)$-log S for simple loading.

TABLE 22
COMPARISON OF RANDOM LOADING TEST RESULTS

	PSEUDO-RANDOM		REPEATED-BLOCK	
TEST SERIES [a]	MEAN FRACTURE LIFE (LOAD REPS.)	MEAN INITIAL STIFFNESS MODULUS (1,000 PSI)	MEAN FRACTURE LIFE (LOAD REPS.)	MEAN INITIAL STIFFNESS MODULUS (1,000 PSI)
A	26,500	263	15,800	245
B	13,500	250	9,600	241
C	8,600	247	11,200	258

[a] Series A; 10% at 128.5 psi, 30% at 113.5 psi, 60% at 98.5 psi.
Series B; 25% at 128.5 psi, 50% at 113.5 psi, 25% at 98.5 psi.
Series C; 60% at 128.5 psi, 30% at 113.5 psi, 10% at 98.5 psi.

Although this work is somewhat limited, it presents an extremely simple method of using simple-load tests to estimate fatigue life in compound random-type loading.

SUMMARY

In summarizing the laboratory and field studies relative to fatigue, no attempt is made to quantitatively compare results, because test methods and procedures of analysis vary widely. The primary objective of this summary is to describe, qualitatively, the general consensus which has been developed somewhat independently of the variation in test methods.

The following factors appear to be significant as regards fatigue of asphaltic concrete;

1. Both laboratory and field investigations indicate that asphaltic concrete responds to repetitive loading in a manner similar to that found in elastic materials; i.e., metal, wood, portland cement concrete, etc. Thus, it appears to be well established that asphaltic concrete does exhibit fatigue properties in laboratory tests as well as in field performance.

2. A linear relationship exists between the log of stress or the log of strain and the log of repetitive loads to failure.

3. The mode of laboratory testing, as between stress and strain controlled, and the failure criteria will influence the results of laboratory tests. In general, constant stress-type tests will respond with an increasing fatigue life to any material or mix property which increases the stiffness of the asphaltic concrete. For example, lower asphalt penetrations, lower void content, and lower temperatures generally result in a higher stiffness modulus and a greater number of cycles to failure. For constant strain tests, the effect of stiffness modulus, asphalt penetration, etc., is reversed. For tests at very low temperatures (approximately 32 F) the fatigue life is unaffected by mode of testing. Conceivably, the difference in test results could be due to the rate of crack propagation and the amount of damage sustained by the specimen. In constant stress tests the load is unchanging during the life of the specimen, causing increased damage with repetitions. In constant strain, the load is reduced to compensate for damage. Also, if the failure criterion is based on the appearance of cracks, the stiffer mixes will tend to propagate cracks faster once the cracks have been formed.

4. The relationship between aggregate gradation and fatigue performance has not been definitely determined. Nijboer has suggested that a minimum ratio (volume basis) between filler (minus No. 200 sieve) and asphalt will result in longer fatigue life in sheet asphalt mixes. Monismith compared an open-graded mix (100 percent passing ⅜ in., 0 percent passing No. 50 sieve) with a dense-graded mix (100 percent passing ½ in., 9 percent passing No. 200 sieve) in constant stress and found the dense-graded (relatively high filler-bitumen ratio) mix to have better fatigue properties. More information is needed to better define the gradation effect; however, on the evidence available, it appears that gradation will be important only as it influences the volume concentration of aggregate and the stiffness modulus.

5. Jimenez et al. indicate that a rough surface textured material is superior to a smooth surface textured aggregate and that an optimum asphalt content is associated with each surface type. Monismith's data tend to support the superiority of rougher (crushed granite) surface textured material; however, he found no optimum asphalt content, rather a continuous improvement in fatigue properties with asphalt contents up to 7 percent in a well-graded (¾-in. maximum size) aggregate.

6. One of the most useful coefficients from the fatigue research appears to be the stiffness modulus. This coefficient can be obtained in a number of ways, as discussed in Chapter Two. Deacon (*25*) and Hicks (*55*) have shown that the stiffness modulus is stress dependent when measured by simple beam flexure; therefore, an evaluation of this factor is necessary for application to the design of the asphaltic mixes.

7. On the basis of laboratory test results, higher bitumen contents have generally resulted in better fatigue properties, particularly when using a rough surface textured aggregate.

8. Fatigue properties, as measured in the laboratory, exhibit considerable variability, and methods of testing and interpretation must take this into account.

9. Rate of loading may have some effect on fatigue behavior, at least when tested above 30 applications per

minute. Deacon has suggested that residual stresses in higher rates of loading tests tend to reduce the fatigue life.

10. Load duration affects fatigue behavior, with longer duration associated with reduced fatigue life.

11. In the process of performing repetitive loading tests, the change in stiffness modulus, deflection, or modulus of rupture can be used to measure fatigue damage. A higher rate of damage appears to occur with the first 10 percent of the repetitive loadings, with a relatively constant and somewhat reduced rate for the next 80 percent of the loadings, followed by an abrupt change to failure.

12. Both the tensile stress and tensile strain have been suggested as a damage determinant. It appears that either factor can be used, providing field loading conditions are satisfied. This is discussed in detail in the next section.

13. Fatigue data indicate, all other factors being approximately equal, that the fatigue service life increases as the void content decreases.

14. The general relationship between the cycles or repetitions of load to failure expressed in terms of the applied strain is given by

$$N = K\,(1/\epsilon)^n \tag{39}$$

in which

N = cycles to failure;
K = a constant depending on the properties of the mix (e.g., void content, asphalt consistency, etc.);
ϵ = magnitude of the applied strain; and
n = slope of line representing linear relationship between log of applied strain and log of cycles of loading.

15. Based on the effort of Deacon, the following tentative conclusions for compound loading are indicated:

(a) Based on two-level sequence testing, stress history has little effect on the rate of damage occurring in fatigue tests.
(b) Repeated-block loading can be used in lieu of pseudo-random loading to estimate fatigue life in compound loading.
(c) Tentative analytical equations are given which can predict compound-loading test results on the basis of simple-loading fatigue tests.

DESIGN IMPLICATIONS

Based on the research described in the previous section, it can reasonably be concluded that asphalt and asphaltic mixes of a wide range of types are subject to fatigue damage similar to that found in most elastic solids. This research, to be meaningful, should serve as a basis to the engineer for specifying material properties, asphaltic mix requirements, and construction standards which will minimize damage due to repetitive loading. Research to date appears to be sufficiently advanced to provide guidelines to the engineer, but not sufficiently developed to be definitive in mix design criteria or construction specifications.

To manipulate the fatigue characteristics of asphaltic mixes, it would be desirable to have a single parameter whose association with fatigue is known. At present the stiffness modulus previously discussed and described seems to be most logical, inasmuch as a considerable amount of information has been reported relative to its correlation with fatigue life.

For the purpose of the following general discussion, stiffness modulus is used as the primary property of the mix to be associated with fatigue. No attempt is made in this section to stipulate how the stiffness modulus should be measured, although specific methods are referred to in order to make specific interpretations and applications.

To use the stiffness modulus as an indicator of fatigue properties, it is first necessary to establish a correlation between these two factors. Unfortunately, this correlation depends on the mode of testing (stress or strain), as indicated in the previous section. Table 23 summarizes the effects of those factors influencing the fatigue properties of asphaltic mixes, according to the type of test performed.

Examination of laboratory data indicates that there is an absence of information as regards interaction of those factors which affect stiffness. For example, if aggregate gradation were adjusted to result in a denser mix, an increase in the stiffness would be expected. If the asphalt penetration were simultaneously increased, a decrease in the stiffness would be expected. The net effect on the fatigue life, however, is not always known. Stated in a different way, two specimens of asphaltic concrete mixes with the same stiffness could have different fatigue properties due to variations in aggregate gradation, asphalt grade, void content, etc. On this basis, the only reliable way to determine the fatigue life is to perform fatigue-type tests. Procedures developed by van der Poel and modified by Heukelom and Klomp can be expected to give some indication of net effect; however, the reliability of this relationship needs further confirmation.

This rather negative conclusion should not be interpreted as completely negating the value of the stiffness modulus. For a specific set of guide specifications wherein the aggregate properties (i.e., gradation, stability, aggregate durability, etc.) are defined, it is highly probable that stiffness can be correlated with fatigue properties. No specific attempt has as yet been reported to verify this possibility, hence a positive determination is beyond the scope of this report.

To pursue the matter of the feasibility of using a stiffness modulus for specific mix design criteria, computations of required stiffness have been made on the basis of deflection criteria suggested by Hveem (*66*) and from the AASHO Road Test (*57*). If for particular deflection and material requirements a limiting stiffness can be established, it would seem possible that maximum or minimum stiffness values could be established for a mix design.

Results of the AASHO Road Test indicate that surface deflection measured with a Benkelman beam can be used to predict the life characteristics of a pavement structure. Although the pavement performance criterion on that project was defined as riding quality, careful examination of the criterion and the basic measurements tends to indicate that a significant amount of the drop in riding quality must have been due to the longitudinal roughness associated with fatigue cracking. The formula used to associate spring deflection to allowable traffic on the AASHO Road Test was

$$\log d = \frac{9.40 + 1.32 \log L_1 - \log W_{2.5}}{3.25} \tag{40}$$

in which

L_1 = single-axle load, in kips;
d = normal deflection, in 0.001 in., measured under a wheel load equal to $L_1/2$.

Thus, the allowable deflection under a 9-kip wheel load for one million applications of an 18-kip single-axle load would be 0.036 in. Hveem (*66*) indicates that a deflection of 0.017 in. under a 7.5-kip wheel load is allowable for heavily trafficed highways.

Using the three-layer theory, it is possible to estimate, by iteration, the required stiffness modulus of the surface layer from

$$\Delta = \frac{a\,p}{E_1} F \tag{41}$$

in which

Δ = deflection of the center of the loaded area;
a = radius of the loaded area;
p = surface loading pressure;
E_1 = Young's modulus (stiffness) of the upper layer; and
F = deflection factor, determined from Table 19.

For the AASHO Road Test, it is possible to compute thickness requirements, in terms of surface, base, and subbase, for a given level of deflection in accordance with relationships given in Ref. (*57*). Information relative to the stiffness moduli for the various materials was obtained from Seed et al. (*141*), Skok and Finn (*146*), and Vesic (*163*). For the deflection criterion suggested by Hveem, it was assumed that thickness requirements obtained by California design procedures (*18*) would also satisfy the deflection requirements suggested by Hveem. In this case, 0.017-in. deflection was used for both the 4- and 5-in. surface thicknesses.

Tables 24 and 25 summarize the results for stiffness requirements based on deflection. It becomes rather apparent that interpretations placed on the stiffness value are complicated by several factors, but most especially by the modular ratios, K_1 and K_2, and the relative thickness of the surface to aggregate base layers. The values presented from the AASHO Road Test results indicate that stiffness modulus values ranging from 60,000 to 1,300,000 psi at 40 to 50 F would be satisfactory, depending on the properties of the subgrade and base and on the thicknesses incorporated in the design. The calculations are inconclusive, due in part to a lack of specific information as to the values of the moduli for the aggregate and the subgrade layers. Three important observations can be made, as follows:

1. The general level of required modulus for the asphaltic surfacing is decreased as the surface thickness decreases.

2. For the more realistic values of aggregate modulus (28,000 psi or less), the stiffness modulus of the surfacing ranged from 450,000 to 1,300,000 psi on the AASHO Road Test and 670,000 to 1,220,000 psi for the California designs.

TABLE 23
FACTORS AFFECTING FATIGUE OF ASPHALTIC MIXES

FACTOR	EFFECT ON STIFFNESS	EFFECT ON FATIGUE LIFE: STRESS CONTROLLED	EFFECT ON FATIGUE LIFE: STRAIN CONTROLLED
Asphalt penetration	Increases with decrease in penetration	Increases	Decreases
Asphalt [a] content	Increases with increase in asphalt content	Increases	Decreases
Aggregate type	Increases with increased roughness and angularity	Increases	Decreases
Temperature [b]	Increases with decreasing temperature	Increases	Decreases
Void content	Increases with decrease in voids	Increases	
Aggregate gradation	Increase from open to dense gradation	Increases	Decreases [c]

[a] Within reasonable limits above laboratory optimum asphalt content as determined from stability tests.
[b] Approaches upper limit for temperatures below freezing.
[c] No significant amount of data; however, seems reasonable on basis of stiffness modulus effect and data obtained in constant stress.

3. To evaluate stiffness modulus requirements reliably, determinations of the aggregate and subgrade moduli are required. Although this makes the problem more difficult, it poses no great obstacle to the analysis of fatigue performance. Seed et al. (*142*) have described procedures for measuring the moduli of untreated materials for use in theoretical analysis of pavement performance.

In summary, it appears that, for temperatures of 40 to 50 F, asphaltic concrete should have a stiffness modulus of approximately 1,000,000 psi. However, the reliability of this value is not considered sufficiently established for mix design criteria.

To interpret laboratory fatigue data in terms of possible significance to asphaltic surfacing performance, it is first necessary to attempt to relate the laboratory test with the in-service pavement. At the present stage of technology, it seems premature to attempt to establish any precise relationship of cycles to failure in the laboratory to load applications for the in-service pavement. Of considerable value, however, may be the ability to order the relative performance (fatigue life) of different asphaltic mixes, to increase or decrease the fatigue life by adjusting the mix composition, and to establish the minimum level of laboratory fatigue life required for a given design situation. To do this by laboratory tests, it is necessary to understand the significance of the results obtained by both constant stress and constant strain type loading.

Assuming that asphaltic concrete acts elastically under dynamic loads at relatively small levels of strain (less than

TABLE 24

CALCULATED STIFFNESS MODULUS OF ASPHALTIC CONCRETE TO SATISFY AASHO ROAD TEST DEFLECTION CRITERIA FOR SPRING CONDITIONS

SOURCE	SUBGRADE MODULUS [a] (PSI)	AGGREGATE THICKNESS (IN.)	AGGREGATE MODULUS (PSI)	ASPHALTIC SURFACING THICKNESS (IN.)	ASPHALTIC SURFACING MODULUS (PSI)	K_1 [b]	K_2 [b]
AASHO	2720	14	54,400	5	500,000	9.2	20
Road		14	27,200	5	1,300,000	48	10
Test	4000	14	40,000	5	300,000	7.5	10
		14	20,000	5	530,000	26.5	5
		14	18,000	5	670,000	37	4.5
		24	40,000	3	60,000	1.5	10
		24	20,000	3	450,000	22.5	5

[a] As suggested in various research publications.
[b] $K_1 = E_1/E_2$; $K_2 = E_2/E_3$.

TABLE 25

CALCULATIONS FOR STIFFNESS MODULUS OF ASPHALTIC CONCRETE TO SATISFY DEFLECTION REQUIREMENTS SUGGESTED BY HVEEM [a]

SUBGRADE MODULUS [b] (PSI)	AGGREGATE THICKNESS (IN.)	AGGREGATE MODULUS (PSI)	ASPHALTIC SURFACING THICKNESS (IN.)	ASPHALTIC SURFACING MODULUS (PSI)	K_1	K_2
4,500	24	45,000	5	740,000	16.5	10
($R = 15$)	24	36,000	5	1,130,000	31.4	8
10,000	16	50,000	4	200,000	4	5
($R = 30$)	16	30,000	4	590,000	19.6	3
	16	24,000	4	1,220,000	51	2.4
15,000	16	30,000	4	280,000	9.3	2
($R = 37$)	16	17,000	4	670,000	39.4	1.2

[a] Ref. (*65*). [b] Estimated.

10^{-3} in./in.), there would be a constant relationship between stress and strain. If this is the case, the results obtained by laboratory tests under constant stress and under constant strain loading would be the same. Pell found this to be true for low-temperature tests in some of his early work, where he was able to convert stress values into strain, and, by so doing, eliminate time and temperature as variables. However, as more data were acquired, it was found that conversion of constant stress test results to strain did not duplicate constant strain test results. The explanation which seems most acceptable for this difference is the difference in the rate of crack propagation previously discussed.

Hicks (*55*) has attempted to evaluate the applicability of the controlled stress and strain tests on the basis of computations of elasticity applied to a three-layer pavement. He assumed two hypothetical pavement sections with a total thickness of 13 and 26 in. and a range of surface thickness of 1 to 9 in. He assigned a stiffness modulus range of from 100,000 to 4,000,000 psi and fixed the modulus of the second layer (base) at 12,000 psi and of the third layer (subgrade) at 3,000 psi. Computations were based on a uniform surface load of 70 psi over a 5-in. radius. Figures 99 and 100, summarizing the results of computations for tensile strain in the under side of the surface layer, show that tensile strain in the 1-in. surface layer is relatively unaffected by the stiffness modulus, E_1. Although there is a pronounced curvature in the figure, the difference in strain between 100,000 and 1,000,000 psi is relatively small. Thus, on the basis of these theoretical computations it appears that a 1-in. thickness of asphaltic concrete surfacing would, for a given loading, be subjected to constant strain in field-loading conditions, regardless of the total thickness of the pavement and the stiffness modulus of the asphaltic layer. A constant-strain fatigue-type test was, therefore, suggested for thin surface layers.

Hicks also reported computations for stress in the under side of the surface layer using the same assumptions of thickness and material properties. Figures 101 and 102 summarize these results. It is pertinent to consider the slope of the curves for various thicknesses of surfacing. In this instance the thicker surfaces tend to be influenced less by the stiffness moduli than the thin section. The indications are that the thicker sections are subjected to a rela-

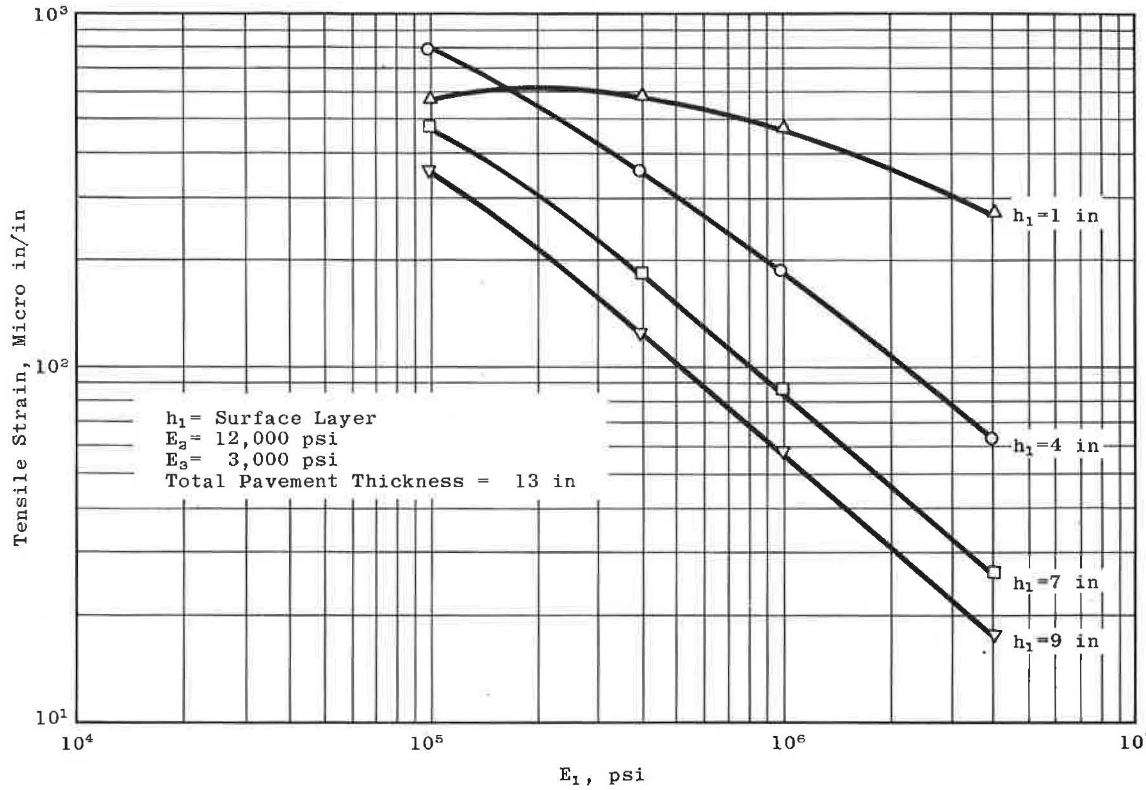


Figure 99. Induced tensile strain as a function of surface modulus, thin section, (after Hicks, 55).

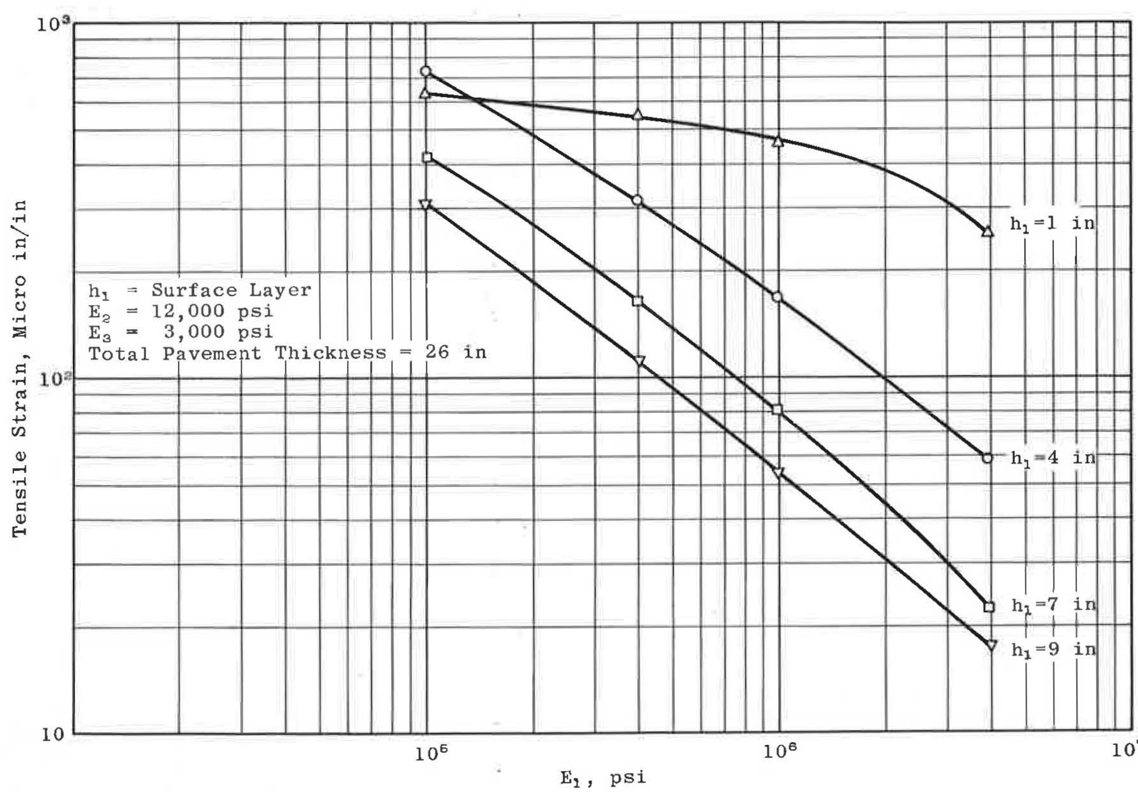


Figure 100. Induced tensile strain as a function of surface modulus, thick section (after Hicks, 55).

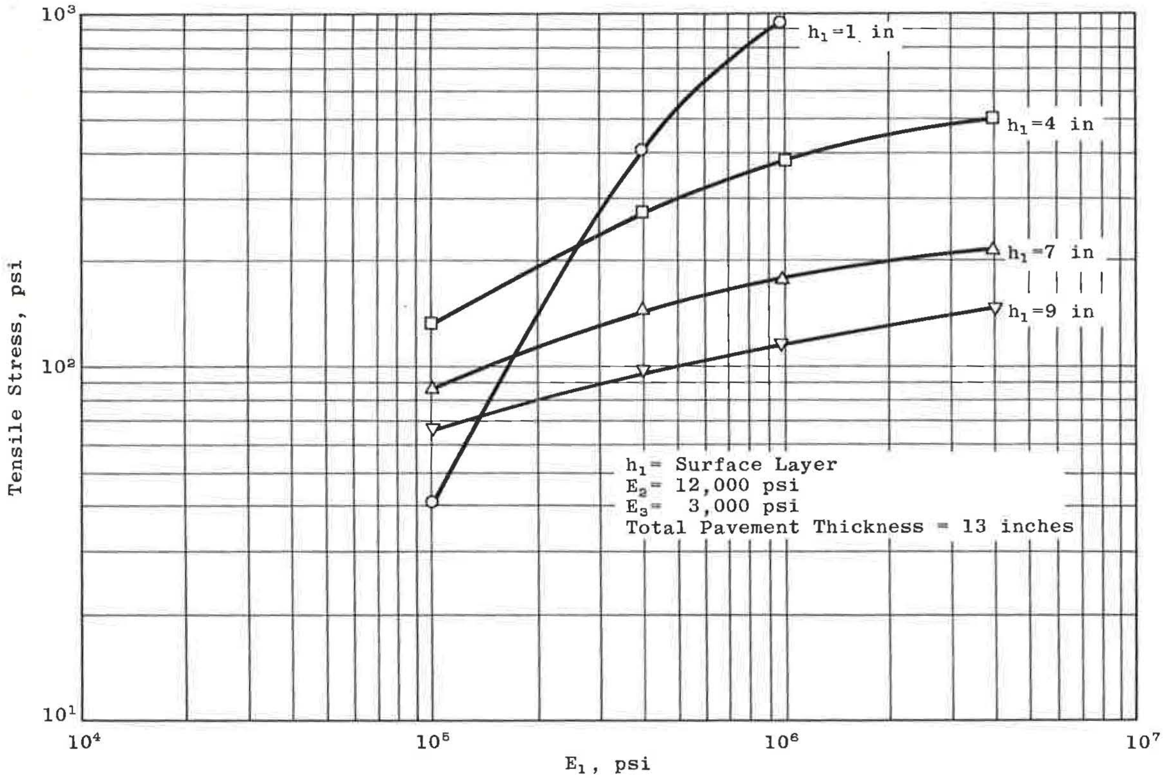


Figure 101. Induced tensile stress as a function of surface modulus, thin section (after Hicks, 55).

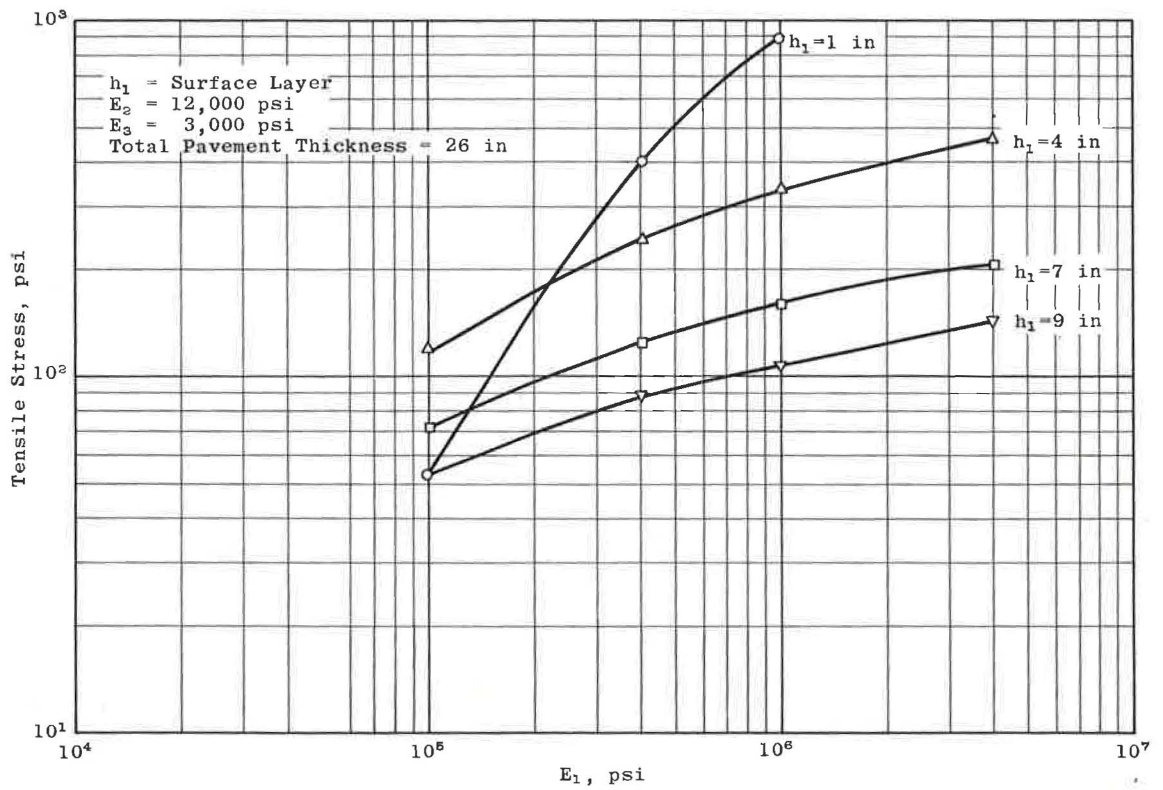


Figure 102. Induced tensile stress as a function of surface modulus, thick section (after Hicks, 55).

tively constant stress, therefore suggesting the constant stress mode of testing.

The results of computations by Hicks, and their interpretation, cannot be considered as proof for establishing the mode of testing to be used. It is interesting to note, however, that the conclusions reached tend to support past experience of highway engineers as to the desirable properties of asphaltic concrete relative to fatigue performance. For example, the general consensus of engineers, as regards improving fatigue properties, indicates a preference for mixes with soft asphalts (high penetration) and possibly with coarse or open gradings. The effect of using such materials and gradings would be to produce a material of relatively low stiffness modulus. For purposes of this interpretation, it must be remembered that in uniaxial laboratory testing, fatigue life, either in constant stress or strain, is indirectly proportional to the level of stress or strain used in the test, and it is assumed this would be the case for in-service surfacing. Therefore, by examination of Figures 99 through 102, it seems reasonable to conclude that the 1-in. surfacing with a low stiffness modulus could have a longer fatigue life than the thicker 4-in. surface layer. Comparison of the stress or strain values for the 1- and 4-in. surface thickness at a stiffness modulus of 100,000 psi indicates higher values of stress or strain for the 4-in. layer. Apparently, for this situation, the thicker layer does not reduce the stress or strain as much as the greater beam depth increases these factors. Hence, for a surface thickness of 1 in., and probably for 2 and 3 in., the lower stiffness modulus would likely result in the best fatigue performance. Considering that the preponderance of experience since the late 1920's has been with pavements of 2- and 3-in. thicknesses, it seems logical to expect a preference for mixes of low stiffness. It is also interesting to compare this interpretation with some of the earliest asphaltic pavements built at the turn of the century. One example is the old "patented"-type pavements constructed in thick layers with very hard asphalts. From reported performance, these pavements had an exceptionally high fatigue life, tending to verify the information available from Figures 99 through 102.

On the basis of these interpretations, hypotheses can be made relative to pavement thickness, stiffness modulus, and fatigue performance, as follows:

1. For thin pavements, asphaltic surfacing should be made with mixes of low stiffness (less than 100,000 psi at critical temperatures) and the fatigue life should be evaluated in the constant strain mode of testing.

2. For thick pavements (5 in. and greater) asphaltic surfacing should be made with mixes of higher stiffness and the fatigue life should be evaluated in the constant stress mode of testing.

The second hypothesis runs contradictory to expressions by other investigators concerned with the problem of pavement design. For example, as the modulus of the asphaltic layer becomes higher and the thicknesses increase, it leads to a situation in which the surface layer is absorbing the greater percentage of the load. This seems to be contrary to the desirable and possibly more economical use of material strength properties expressed by Housel (*62*), who has suggested that a stiff surface layer does not mobilize the strength capability of the remaining pavement elements, particularly the subgrade. To mobilize the maximum strength would require that the pavement deflect several times more than present recommendations for permissible deflection. Fatigue information, with asphaltic mixes in current use, tends to show that it would be necessary to reduce the thickness of asphaltic concrete surfaces to a very thin layer in order to satisfy the requirements suggested by Housel.

Another situation appears to be confusing as regards the hypothesis that a higher stiffness value could produce a higher fatigue life. Specifically, fatigue of asphaltic mixes at high temperatures and the resulting low stiffness values is considered uncommon and, therefore, contradictory to the hypothesis. However, the fatigue life of a given mix is associated with the stress or strain level anticipated for the actual construction (design) and traffic (load) situation. As determined from computations by the three-layer system, the stress in the surface layer is greatly dependent on the modular ratio, E_1/E_2. If the surface and base modular ratio, E_1/E_2, for a relatively thin surface layer is small, theoretically the tensile stress or strain in the under side of the surface layer will approach zero or be in compression, and the fatigue life would probably not be critical as regards performance. This is shown in Figure 103, where it can be noted (Fig. 103c) that for the 3-in. surface thickness the asphaltic layer is estimated to be in compression whenever $E_1/E_2 \approx 1.0$.

There are at least two observations which can be made on the basis of this figure, as follows:

1. Considering the use of untreated aggregate base, with a modulus of 20,000 psi or less, it would be necessary to have a binder and mix which would develop an equal modulus. From a practical consideration, this would seem possible only under very high temperatures or very slow rates of loading. Although this may be one way to avoid fatigue problems, it is suggested that problems of raveling and rutting would develop.

2. Examination of Figure 103 indicates that as E_1/E_2 increases, the tensile stress in the thinner layers increases more rapidly than corresponding stresses in the thick 8-in. layer.

Thus, it might be concluded that if mixes with stiffness moduli approaching the modulus of the aggregate base could be developed, without aggravating problems of rutting and raveling, the damaging effects of fatigue could be minimized.

The second hypothesis suggests still another conclusion which conflicts with general opinion; namely, asphalts of poor durability and which harden rapidly, when used in thick surface layers would outperform the more durable asphalts in terms of fatigue life. Because this implication of the second hypothesis runs so contrary to engineering

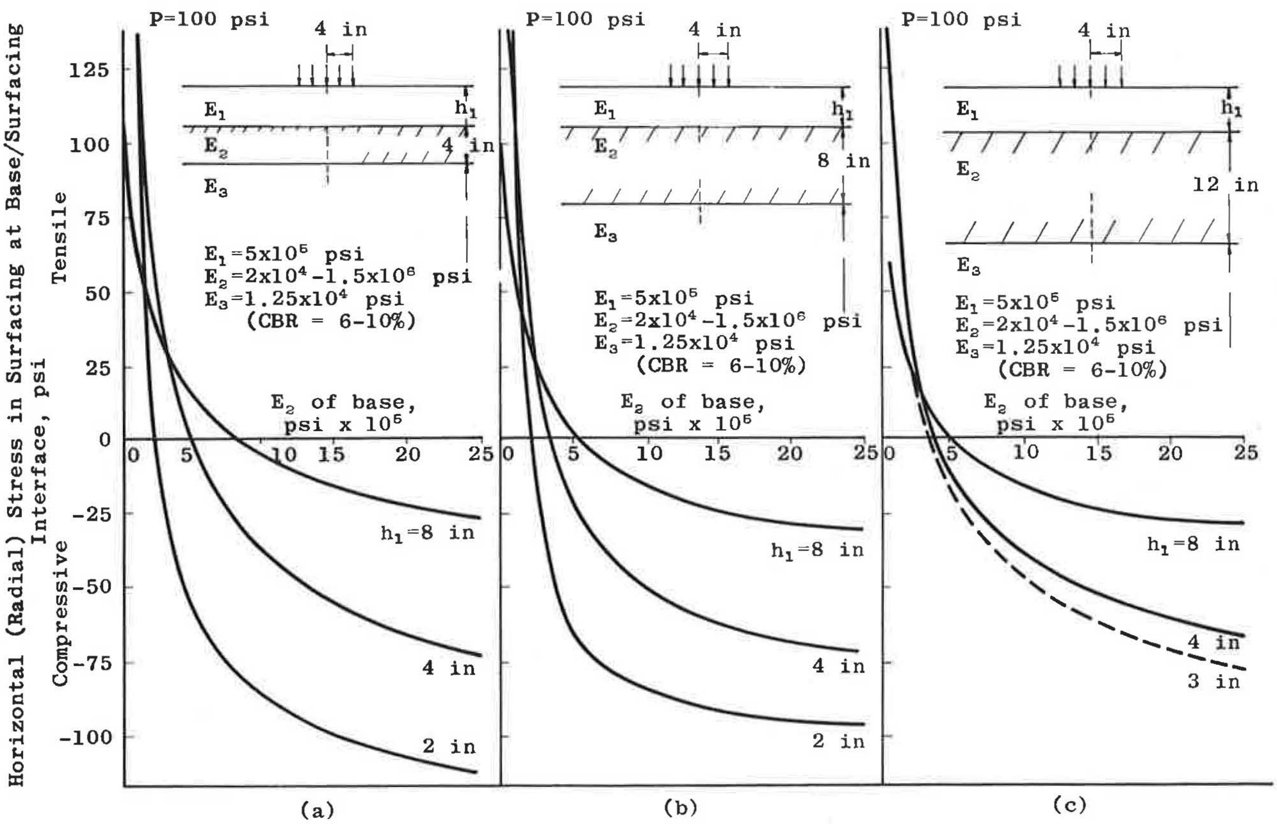


Figure 103. Horizontal stress at underside of surface layer (after Whiffen and Lister, 165).

concepts of asphalt durability, a small test program was carried out to determine if there is any possibility that the hypothesis could be correct.

For the test program, constant stress fatigue tests were made using a control (unweathered) and a weathered asphalt in the asphaltic concrete. The weathered asphalt was produced in the rolling thin film oven (RTFO) developed by the California Division of Highways for their work on asphalt specifications (*70*). The original use of this equipment was to simulate asphalt hardening associated with the production of asphaltic concrete. For this particular testing program, the time in the oven was extended to obtain an asphalt of approximately 15 penetration. Figure 104 shows the results of pilot tests to determine the timing required in RTFO to obtain the required penetration. On the basis of this information, a sufficient quantity of asphalt was processed to manufacture 12 asphaltic concrete test specimens for fatigue testing. For comparison, 12 test specimens were also fabricated with the same asphalt without accelerated weathering. Table 26 summarizes pertinent information relative to the asphalt and the asphaltic concrete used in the tests. The estimated stiffness moduli

TABLE 26

SUMMARY OF STIFFNESS MODULI DETERMINATIONS

	PROPERTIES OF RECOVERED ASPHALT IN FATIGUE SPECIMEN		ESTIMATED STIFFNESS [a]				
ASPHALT DESCRIPTION	PENETRATION	SOFTENING POINT (°F)	RECOVERED ASPHALT (PSI × 10^3)		ASPHALTIC CONCRETE (PSI × 10^5)		MEASURED [b] STIFFNESS (PSI × 10^5)
Unweathered	52	123	2.66[c]	1.42[d]	6.4[c]	2.63[d]	2.00-2.13
Weathered	14	72	13.0	7.80	21.0	9.4	4.76-6.24

[a] Based on test temperature of 68 F and load duration of 0.10 sec.
[b] Deflection-based stiffness modulus.
[c] Van der Poel nomographs.
[d] Heukelom and Klomp solutions and nomographs.

were obtained from the van der Poel and Heukelom and Klomp nomographs in Chapter Two.

Comparison of the estimated stiffness modulus of the asphaltic concrete with the deflection-based stiffness modulus indicates that the procedures developed by Heukelom and Klomp estimated the modulus for the unaged asphaltic mixes within a factor of 1.3 (approximately 30 percent high). These same procedures estimated the modulus of the aged asphaltic mixes within a factor of 1.5 to 2.0. These values would fall within the confidence factor of $\pm$ 2.0 usually claimed for the nomographs. A similar comparison with the van der Poel nomograph does not provide estimates within the factor of $\pm$ 2.0. The difficulty with the van der Poel nomographs is that (a) estimates of asphalt stiffness, for asphalts of negative P.I. (penetration index), are extremely sensitive, with the width of a pencil line being enough to cause large changes in the estimate of the stiffness; and (b) the stiffness of the asphaltic concrete is limited to only one curve for aggregate volume concentration.

Figures 105 and 106 and Tables 27 and 28 provide some definitive information relative to the relationships between stress, strain, and fracture life, and the dispersion found in the cycles to failure from fatigue testing. In consideration of past research, straight lines have been used to represent the continuous relationship of fracture life to stress or initial strain.

To estimate the significance of these test results, it is necessary to estimate fatigue performance for a specific pavement design. For example, estimates of the cycles to failure for a 9,000-lb load on two pavement sections can be made from the following information:

1. Surfacing with unweathered asphalt: $E_1 = 200{,}000$ psi, $E_2 = 9{,}000$ psi, $E_3 = 4{,}500$ psi, $h_1 = 4$ in., $h_2 = 24$ in., $\epsilon = 412 \times 10^{-6}$ in./in. (computed). Estimated cycles to failure (Fig. 106) = 400,000.

2. Surfacing with weathered asphalt: $E_1 = 400{,}000$ psi, $E_2 = 9{,}000$ psi, $E_3 = 4{,}500$ psi, $h_1 = 4$ in., $h_2 = 24$ in., $\epsilon = 208 \times 10^{-6}$ in./in. (computed). Estimated cycles to failure (Fig. 106) = 700,000.

Thus, the estimated fatigue service life would be longer for the mix with the weathered asphalt.

In view of the rather high standard deviation for the fracture life reported in Table 27, particularly at the lower stress levels, an attempt has been made to assign some level of statistical significance to the difference in the foregoing estimates for cycles to failure. For this purpose, the standard deviations corresponding to the 90-psi (unweathered asphalt) and 120-psi (weathered) asphalt were used. The ratio of the difference in these two means (300,000 cycles) to the standard error of the estimate for the difference in the two means (159,000 cycles) is 1.89. For five degrees of freedom, the estimated difference of 300,000 cycles would be considered significant at the 90 percent level but not at the 95 percent level. Thus, this apparently large difference in the estimated cycles to failure must be interpreted with caution as to the real difference which might be expected in actual pavement performance.

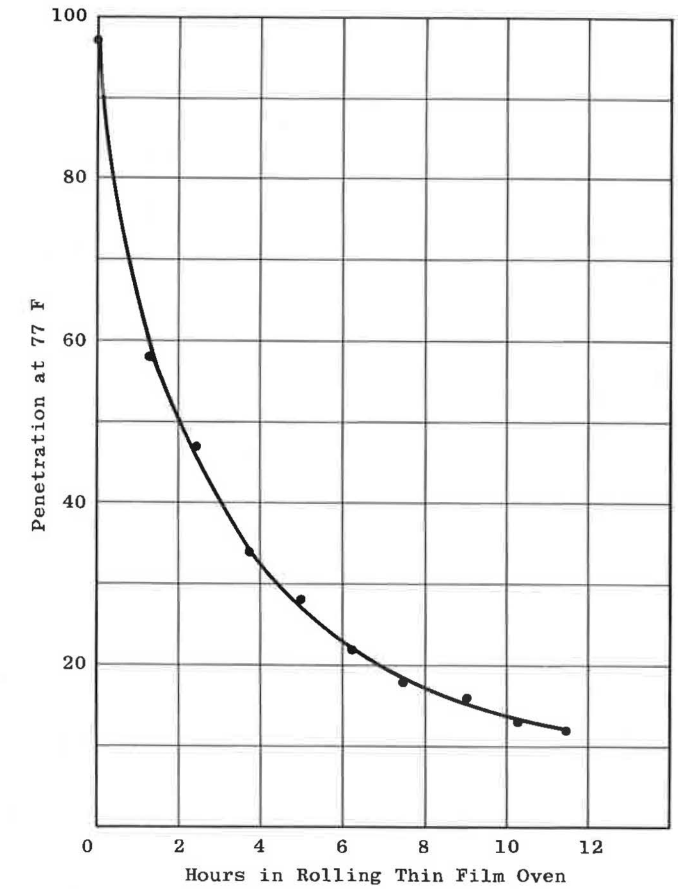


Figure 104. Accelerated weathering of asphalt in rolling thin film oven.

A second example, using thicker asphaltic concrete surfacing, may also be meaningful in view of the current trend in pavement designs. For this example, it has been assumed that the structural section is composed of 6 in. of asphaltic surfacing and 20 in. of untreated aggregate. With these thicknesses, and using the same values of E_1, E_2, and E_3, the following strains and fatigue lives are estimated for the surfacing:

1. Surfacing with unaged asphalt: $\epsilon = 322 \times 10^{-6}$ in./in. (computed). Estimated cycles to failure (Fig. 106) = 1,700,000.

2. Surfacing with aged asphalt: $\epsilon = 145 \times 10^{-6}$ in./in. (computed). Estimated cycles to failure (Fig. 106) = 5,000,000.

This estimated three-fold difference in fatigue life depends on the ability to extrapolate along the indicated straight line in Figure 106. In view of the total research dealing with fatigue, this does not seem too unreasonable, although the exact slope of the line is not well defined from the data., Some flattening of this line at very high cycles may be indicated from past research, although it can reasonably be assumed that both types of asphalt specimens would be so affected. No attempt to assign a level of significance has been made in this instance, because no

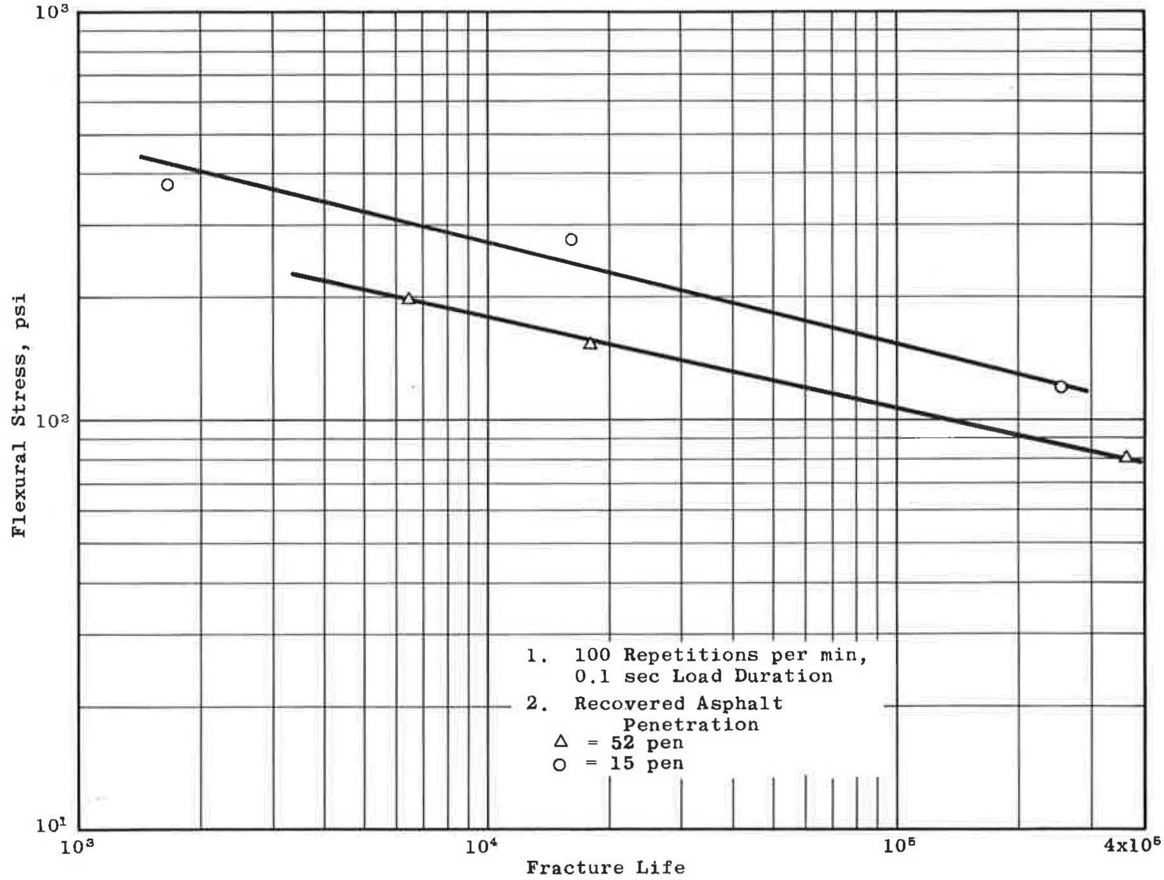


Figure 105. Number of repetitions as a function of flexural stress.

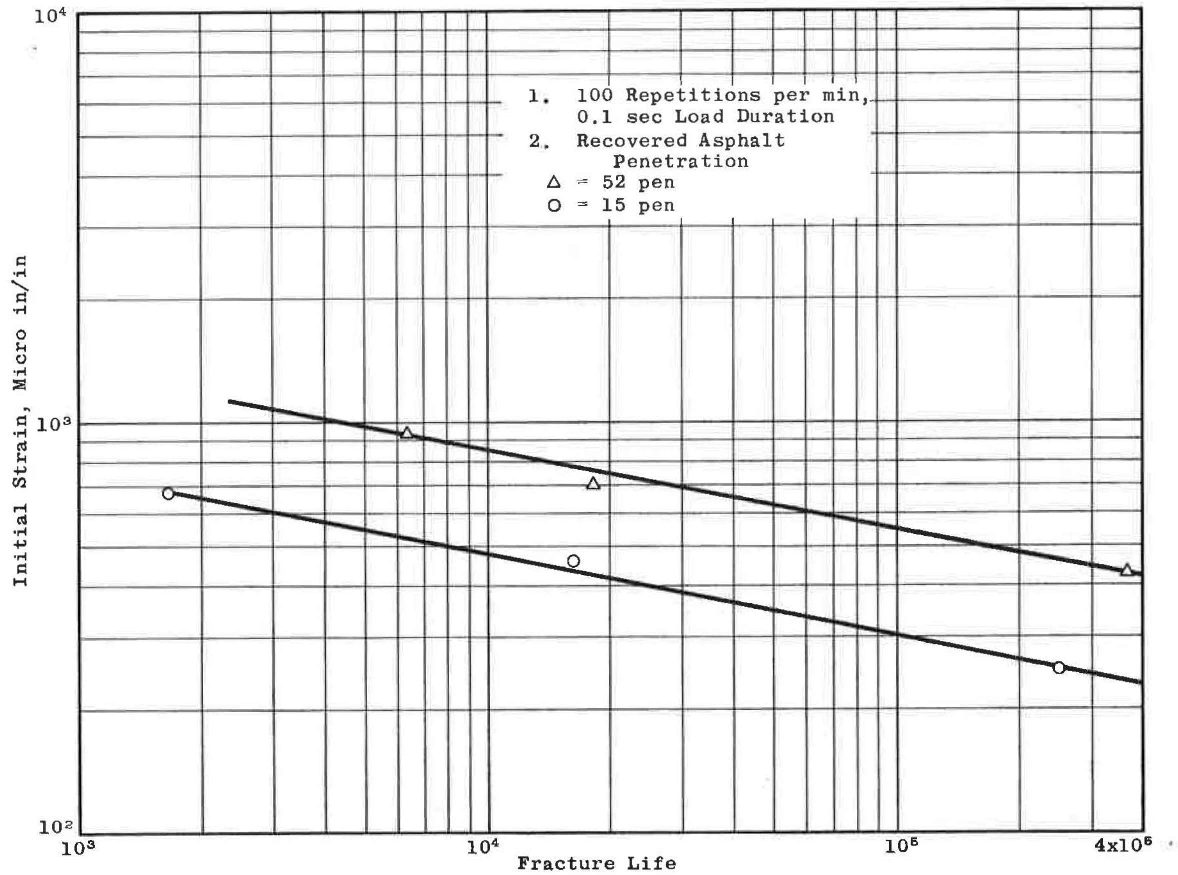


Figure 106. Number of repetitions as a function of initial strain.

TABLE 27

MEAN AND STANDARD DEVIATION OF STIFFNESS MODULUS AT 68 F

SPECIMEN DESCRIPTION	STRESS (PSI)	NO. OF SPEC.	STIFFNESS (PSI) MEAN	STD. DEV.	COEFF. OF VAR. (%)
Unweathered asphalt	200	4	213,000	44,600	20.9
	150	4	200,000	42,800	21.4
	90	4	211,000	39,400	18.6
Weathered asphalt	375	4	582,000	112,000	19.2
	280	4	624,000	131,000	20.9
	120	4	476,000	34,000	7.2

TABLE 28

MEAN AND STANDARD DEVIATION OF FRACTURE LIFE AND INITIAL STRAIN AT 68 F

			FRACTURE LIFE (CYCLES)			INITIAL STRAIN (IN./IN. × 10^{-6})		
SPECIMEN DESCRIPTION	STRESS (PSI)	NO. OF SPEC.	MEAN	STD. DEV.	COEFF. OF VAR. (%)	MEAN	STD. DEV.	COEFF. OF VAR. (%)
Unweathered asphalt	200	4	6,474	4,909	75.8	972	200	20.6
	150	4	17,997	11,438	63.6	717	88	12.3
	90	4	363,025	194,385	53.5	431	77	17.8
Weathered asphalt	375	4	1,665	1,108	66.5	662	109	16.4
	280	4	16,172	9,238	57.1	464	106	22.8
	120	3 [a]	252,926	142,348	56.3	253	24	9.5

[a] Fourth specimen discontinued after approximately 70,000 cycles of loading.

information is available as to the dispersion of the data (standard deviation) at the computed strain levels.

Thus, it seems that asphalts which become hard through aging can demonstrate good fatigue performance when used in thick asphaltic concrete surfacing. Although the data presented herein do not represent the results of an extensive research program and depend on the Rolling Thin Film Oven to produce a weathered asphalt, the results pose a question relative to the significance of weathered asphalt to fatigue performance of relatively thick asphaltic surface layers.

In the previous examples, in which the relative fatigue life was estimated from laboratory data, the initial strain was used as the prime determinant of performance. It is important to discuss why this factor was selected. First, it is believed that some type of constant load test should be used in testing programs to be associated with designs of relatively thick asphaltic layers. Some of the reasons for this have already been discussed. It is also thought that *in-situ* pavements will generally be subjected to constant load conditions. For example, a given highway will be subjected to continuous applications of a range of loads during its planned service life; these loads will not be reduced in order to maintain a constant strain in the asphaltic layer. Thus, constant stress would seem the more logical mode of laboratory testing. Because the simple-beam fatigue test is a uniaxial test, uncomplicated by Poisson effects, it may not be correct to assume that the laboratory stress and the *in-situ* stress will be the same. However, strain represents a physical change which should be directly related as between the pavement and the laboratory. Therefore, initial computed strains, on the assumption of elastic behavior, for a simple beam or from a layered system have been used to estimate the relative cycles to failure for assumed structural sections.

A procedure for interpreting fatigue of asphaltic surfacing suggested by Heukelom and Klomp (*51*) is shown in Figure 107. To illustrate how the relationships of this figure apply, a specific example is considered. The first step would be to estimate the overall thickness and layer thickness requirements by some established procedure considered generally reliable. Second, either measure or estimate the modular values of each material to be used. Values for the subgrade and granular materials can be obtained by repetitive triaxial testing or by established correlations with standard tests. Values for the asphaltic surfacing can be obtained in accordance with procedures for measuring stiffness outlined in Chapter Two. Third, calculate the strain in the under side of the asphaltic surfacing. Two examples of this interpretation are given, as follows:

CASE	*E*, MIX (KG/CM²)	*E*, MIX (PSI)	ϵ (IN./IN.)	EST. FATIGUE SERVICE LIFE[a]
1	92,500	1.3×10^6	0.88×10^{-4}	4.0×10^6
2	35,200	0.5×10^6	1.72×10^{-4}	1.6×10^6

[a] From Figure 107.

Thus, based on a knowledge of the stiffness modulus of the mix and the strain, the fatigue life can be estimated. Using this in reverse would be a way of pavement design. That is, define the fatigue life required and establish thickness combinations and modular values which would provide this life. Further verification of the exact position of the curves is required, and procedures for handling variable traffic loads are necessary. However, the procedure contains all of the necessary parameters.

Work sponsored by Shell Oil Company (*29, 143*) describes a thickness design procedure which includes the fatigue characteristics of asphaltic surfacings. Dormon and Metcalf (*29*) suggest that a temperature of 50 F should be used for the critical determination of strain in the asphaltic surfacing and that a typical modulus at that temperature would be 900,000 psi. Based on work by Monismith and his associates, this appears to be a reasonable value at the specified temperature. There is, however, reason to believe that some sizable variation could occur in this 900,000-psi value. For example (*101*), the stiffness at 40 F ranges from 1,140,000 to 467,000 psi as a function of the asphalt penetration and type of aggregate. It is emphasized that this is a bending modulus and thus is influenced by both the tensile and compressive moduli for the mix. Dormon and Metcalf (*29*) have also suggested that for a stiffness of 900,000 psi (63,400 kg/cm²), the tensile strain in the asphalt should not exceed the following values:

WEIGHTED LOAD APPLICATIONS (EQUIV. 9-KIP WHEEL LOAD)	TENSILE STRAIN IN ASPH. LAYER (IN./IN.)
10^5	2.3×10^{-4}
10^6	1.45×10^{-4}
10^7	9.2×10^{-5}
10^8	5.8×10^{-5}

It should be noted that these are the values obtained from Figure 107.

Figure 108 is a typical design curve of the type used by Shell Oil Company in their 1963 design charts for flexible pavements. Examination of this figure indicates that two criteria have been used for design determination; namely, tensile strain in the asphaltic layer and subgrade compressive strain. Similar criteria have been suggested by other investigators; e.g., Peattie (*115*) and Skok and Finn (*146*). Based on the design curves in Figure 108, tensile strain in the asphaltic layer is not critical to the overall structural design of an asphaltic pavement. Its main function appears to be in setting the minimum thickness of the asphaltic surfacing. In considering these two criteria, it is important to

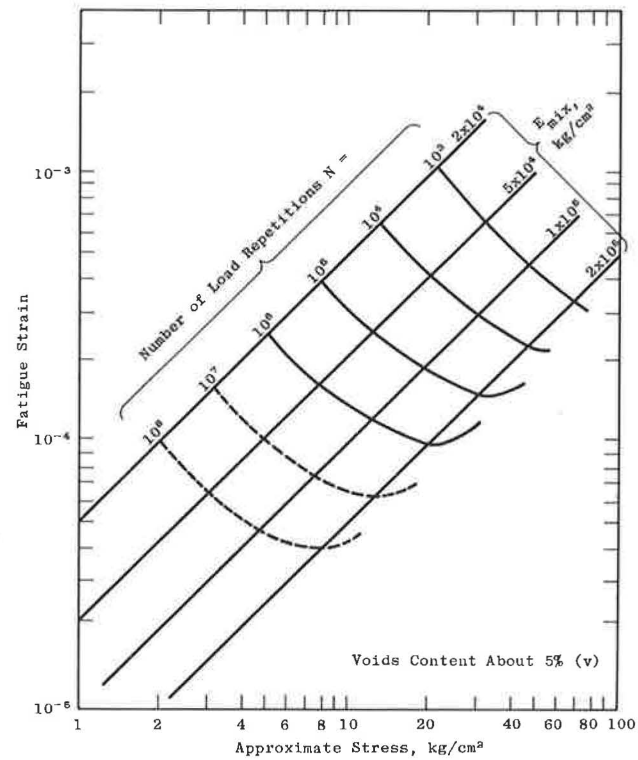


Figure 107. Bending fatigue strain upon repeated loading of bituminous base materials (after Heukelom and Klomp (51) by permission.

realize that for these design curves the critical period for the fatigue of asphaltic concrete is assumed to be during the cold periods and the critical period for compressive strain during the warm periods. To test the validity of this conclusion, a comparison of the fatigue lives in warm and cold weather periods is made. For this comparison, parameters were obtained from Dormon and Metcalf (*29*) and the Shell Oil Company design curves (*143*), as follows: Number of load applications = 10^6, subgrade modulus = 4,500 psi; intermediate layer modulus = 9,000 psi; required thickness design = 4 in. surfacing, 24 in. granular base; $E_2/E_3 = 2.0$. With this as the given information, estimates of fatigue life are made for summer and winter temperatures, as follows:

TIME OF YEAR	SURFACE MODULUS (PSI)	STRAIN IN ASPH. LAYER (IN./IN.)	ESTIMATED FATIGUE LIFE (CYC)
Summer	150,000[a]	490×10^{-6}	0.165×10^6
Winter	900,000[a]	134×10^{-6}	1.0×10^6

[a] Suggested by Dormon and Metcalf (*29*).

Figure 109 was used to estimate the fatigue life of the two surface layers. The basic curve for a modulus of 900,000 psi is from information included in Dormon and Metcalf (*29*). The fatigue properties for a surface layer of 150,000-

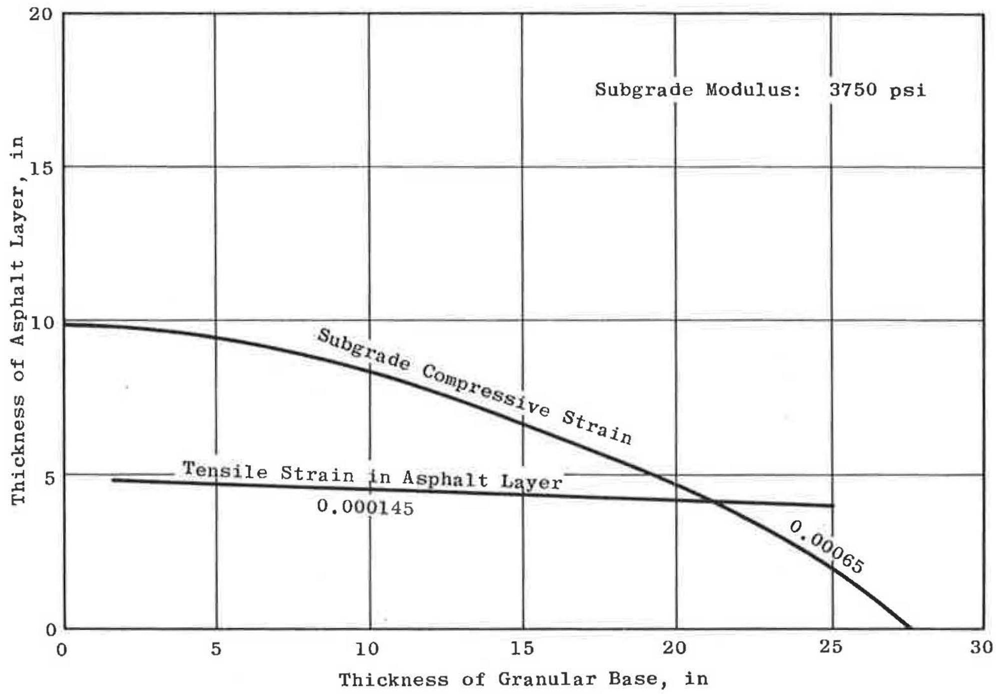


Figure 108. Design curve for flexible pavement (after Dormon and Metcalf, 29).

psi modulus were obtained by extrapolations of Figure 107, which is also included in the work of Dormon and Metcalf.

From the estimates of fatigue life given in the foregoing the damage per load application in the summer may be higher than fatigue damage in the winter. Further complications can arise in this interpretation due to changes in the subgrade modulus, particularly for those geographical areas subject to spring thaw. However, it must be concluded that

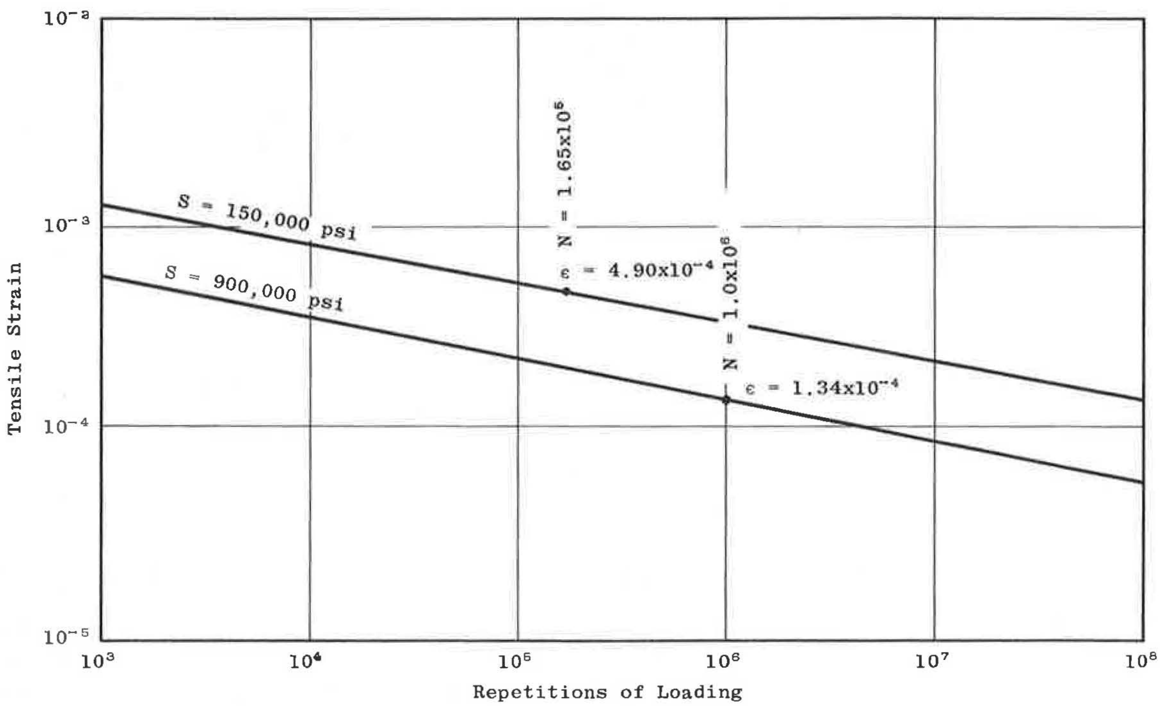


Figure 109. Estimated cycles to failure (adapted after Dormon and Metcalf, 29).

more attention should be given to fatigue of asphaltic concrete during summer conditions. Pell (*120*) has expressed a similar conclusion, as follows:

> It has been shown that tensile strains of a magnitude sufficient to initiate fatigue cracks occur on the road surface and theoretical considerations of a layered system indicate that even greater tensile strains occur on the under side of the top layers. These tensile strains will be a maximum when the overall stiffness of the entire structure is a minimum. The stiffness of bituminous materials is dependent on temperature and the critical condition is therefore likely to arise at high temperatures during the summer months.
>
> However, the fatigue tests at high temperatures show that although cracks initiate under these conditions, they propagate only slowly due to the lower stress, and thus failure will not necessarily be apparent at this time. But once the temperature falls and the stiffness of the bituminous layers increases, there will be an increase in the stress, particularly at the tip of the crack, owing to stress concentration effect. This will result in more rapid propagation of any fatigue cracks under winter conditions, but again it will not necessarily lead to failure owing to the freezing of the subbase and subgrade and the resultant increase in strength. During the thaw period, however, the fact that the surface layers are cracked increases greatly the likelihood of pavement deterioration from penetration of water and consequent local subgrade failure.

In summary, it seems reasonable to believe that fatigue properties of asphaltic concrete may soon be incorporated into asphaltic mix design requirements and made a basic part of thickness design procedures. It appears that to be meaningful it will be necessary to associate fatigue properties in terms of stress and strain in the actual pavement construction to stress and strain measurements in the fatigue test. To do this, it is apparent that it will be necessary to incorporate the properties of the underlying materials into the stress and strain analysis of the actual pavement. Thus, at the present state of technology the greatest need is to develop field correlations between laboratory fatigue properties and performance of asphaltic surfacings.

CHAPTER SIX

DURABILITY OF ASPHALTIC SURFACES

Durability of asphaltic concrete has previously been defined herein as the long-term resistance to the effects of aging. Specifically, for asphalt and aggregate, *per se*, durability usually refers to the rate of change of the physical properties with time. For asphaltic concrete, good durability can be described as the apparent ability to provide long-term performance without abnormal amounts of cracking and raveling. An asphaltic surface could conceivably be composed of asphalt and aggregate, each completely unaffected by time, but because of poor mix design or construction would not be resistant to the abrasive action of traffic. This asphaltic surface would have poor durability even though the asphalt had good durability. Or, conversely, an asphaltic mix could be made with an asphalt which hardens rapidly with time but, by means of adjustments in mix design and construction control, would give acceptable performance. This surface would be considered to have good durability even though the asphalt has poor durability as measured by conventional tests. For purposes of this chapter, the durability of the surface is considered to be of prime importance, whereas the durabilities of asphalt and aggregate are important only insofar as they affect the overall performance. An attempt is made to review the available information on durability of asphalt, aggregate, and asphaltic mixes, and to determine if there is a consensus as to present ability to identify material characteristics which will distinguish between the durable and nondurable behavior of mixtures.

The fact that asphaltic mixture durability plays a key role in the performance of asphaltic surfacing appears to be well documented in the literature. Hveem (*68*) has divided pavement deterioration into three failure categories, as follows: (1) deformation caused by traffic; (2) cracking due to effects of traffic and material properties; and (3) disintegration due to traffic, material properties, and environment. Thus, material properties are listed as major contributors to pavement deterioration. It is with these properties that this chapter is primarily concerned.

It is pertinent to point out that this summary of durability deals primarily, but not exclusively, with investigations reported since the end of World War II. It was generally considered by the authors that work prior to that time would probably not be applicable to modern mix design or construction procedures. Also, it was not possible to review every article published on the subject of durability, particularly the general area of "hardening"* of asphalt; the volume of available literature was simply too great. For those who desire to pursue the subject in detail, Refs. (*10, 56*) provide an extensive listing of pertinent research.

* "Hardening" has been used by technologists as a general term implying an undesirable change, which occurs with time, usually as regards a reduction in penetration or ductility. Although the term is not entirely satisfactory, largely because it has no real meaning, it is used herein in reference to past research. However, wherever possible, specific terminology is substituted.

ASPHALT DURABILITY

Vallerga, Monismith, and Granthem *(162)* state that the characteristic deficiencies of nondurable asphalt are "disintegration and fracture." To avoid disintegration, these investigators have suggested that "asphalts should have adequate adhesive and cohesive qualities and be of such quality that detrimental changes in these properties will not occur with time in service." Fracture was defined as the breaking or cracking due to shrinkage and brittleness of the surfacing. For both fracture and disintegration, the resistance to hardening of the asphalt was described as being of critical importance. According to these authors, a number of factors can contribute to this hardening. Specifically, the factors (not necessarily in order of importance) are:

1. Oxidation.
2. Volatilization.
3. Polymerization.
4. Thixotropy.
5. Syneresis.
6. Separation.

There is considerable argument among asphalt technologists as to the exact process and dominant factors influencing the weathering (aging) of asphalt. Terminology also varies; however, the six items suggested seem to be representative of the general categories referred to by most investigators. No attempt is made herein to evaluate the technical differences between various technologists. It appears to be more important, from the mix design point of view, to be aware of the way in which asphalts age or weather with time, and to consider ways of controlling the rate of aging through design and construction criteria.

Vallerga et al. *(162)* define the six factors related to deterioration as follows:

1. Oxidation is the reaction of oxygen with asphalt, the rate depending on the character of the asphalt and the temperature.
2. Volatilization is the evaporation of the lighter constituents from asphalt and is primarily a function of temperature.
3. Polymerization is a combining of like molecules to form larger molecules, causing a progressive hardening.
4. Thixotropy is a progressive hardening due to the formation of a structure within the asphalt over a period of time which can be destroyed to a degree by reheating and working the material. Thixotropic hardening is generally associated with pavements which have little or no traffic.
5. Syneresis is an exudation reaction in which the thin oily liquids are exuded to the surface of the asphalt film. With the elimination of these oily constituents, the asphalt becomes harder.
6. Separation is the removal of the oily constituents, resins, or asphaltenes from the asphalt as caused by selective absorption of some porous aggregates.

Traxler *(152)* has suggested nine additional factors, including several effects of light, water, chemical reaction with aggregate, microbiological deterioration, and adsorption of heavy asphalt components on the surface of the aggregates.

The rate at which these, or possibly other reactions not specifically identified, occur appears to be extremely complicated and has been argued at length in the literature. For the engineer, these arguments may be superfluous because each reaction seems to lead to undesirable change or embrittlement of the asphalt, which in turn has been associated with asphaltic concrete of poor durability properties. The subsequent literature review describes the results of research associated with the hardening of asphalt and some of the consequences of the hardening in terms of the performance of asphaltic surfacings. In this regard, it is pertinent to examine some of the early research dealing with asphalt hardening.

As early as 1897 Dow indicated that stability (durability) of the asphalt, when exposed to high temperatures, is important to the satisfactory performance of pavements *(83)*. At that time he suggested that, for 50 to 120 penetration asphalts, the retained penetration should be not less than 75 percent after weathering for 30 hr at 400 F. Thus, it seems apparent that technologists have recognized for some time the desirability and need to retain certain physical properties of asphalt in order to obtain a desired performance.

Lewis and Welborn *(83)* have described a series of investigations to develop a standard test which could be used to measure the retained properties of asphalt after plant mixing and field service. The Lewis thin-film oven test was subsequently proposed for use in evaluating asphalt durability. To develop a standard aging test, these investigators first had to decide on methods of evaluation, or tests, which could be related to field observations and performance. Lewis and Welborn chose the penetration, softening point, and ductility tests, probably because most of the research up to that time used these tests. Figures 110 and 111 were

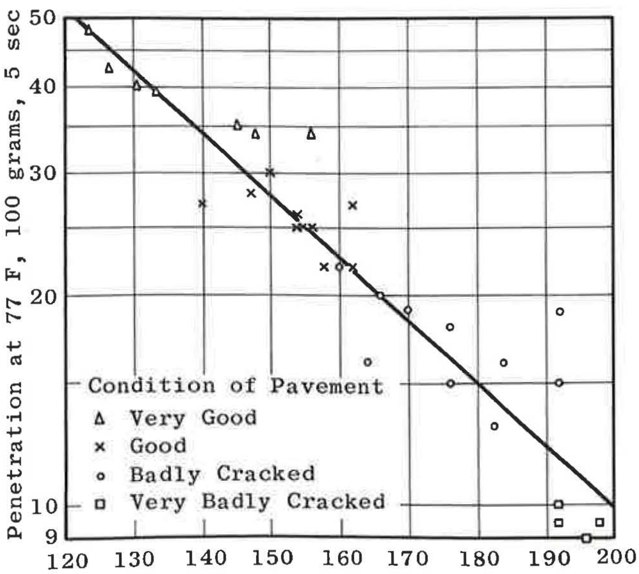


Figure 110. Relation between softening point and penetration of recovered bitumen compared with condition of Detroit pavements (after Lewis and Welborn, 83).

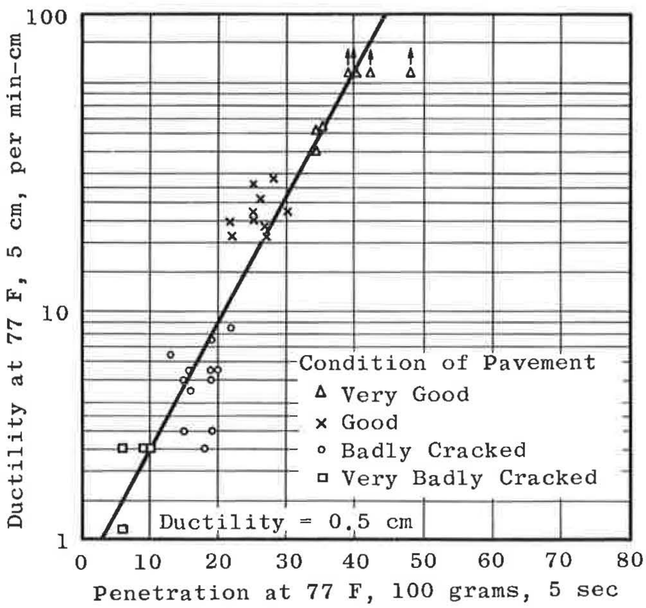


Figure 111. Relation between penetration and ductility of recovered bitumens compared with condition of Detroit pavements (after Lewis and Welborn, 83).

typical of available information. The basic data for these curves were originally published by Shattuck. At that time, performance information was associated with sand-asphalt mixes manufactured with 40-45 and 50-55 penetration asphalt. On the basis of available information at the time, and based on field performance, cracking could be expected when the recovered asphalt had a penetration value less than 20, a ductility of less than 10 cm, and a softening point greater than 150 F.

Hubbard and Gollomb (*63*) have suggested the following five rules for asphaltic mix design necessary to insure long life by preventing cracking and disintegration in an asphaltic pavement or wearing course due to hardness of the asphalt:

1. Use as soft an asphalt as possible, without reducing stability below the minimum required to prevent displacement under traffic.
2. Use as high a percentage of asphalt as possible, without reducing stability below the minimum required to prevent displacement under traffic. In this way film thickness is increased to the maximum practical extent and air permeability is decreased.
3. At paving plants, prepare mixtures at the lowest practicable temperatures.
4. Compress all asphaltic paving mixtures thoroughly, so that they will be as impermeable to air as possible.
5. Seal coat all compressed mixtures which are readily permeable to air, thus preventing free circulation of air within the mix.

Relying largely on reports of other investigators, Hubbard and Gollomb suggested that crack expectancy may be related to asphalt penetration as follows:

1. Asphalt penetration of 20 or less, bad cracking.
2. Asphalt penetration of 30 or more, no cracking.
3. Asphalt penetration between 20 and 30, uncertain.

Lewis and Welborn (*82*) attempted to relate the effects of asphalt hardening to pertinent physical properties of asphaltic mixtures. Some of the conclusions reported from this investigation are:

1. The physical properties (compressive strength, Hubbard-Field stability, flexural strength, and abrasion) of a given mix are dependent on the consistency of the contained asphalt at the test temperature.
2. Because the data from several investigations show that the bitumens extracted from pavements of the type that tend to crack usually have low penetrations and ductilities at 77 F, it would seem logical to make physical tests on asphaltic mixtures at low temperatures, in the range corresponding to these penetrations and ductilities, and to correlate the results with the properties of the contained asphalts at these temperatures.
3. Because durability of asphalt in bituminous mixtures is dependent on changes occurring in the asphalts during mixing and subsequent weathering, those asphalts that have the greatest resistance to change should be the most durable. This study indicates that changes in the asphalt contained in a mixture can be measured directly by tests on the recovered bitumen, or indirectly by changes in the physical properties of the mixture.
4. The durability of asphaltic pavements as influenced by the consistency of the asphalt depends on the climatic environment in which they are located. The critical penetration of the bitumen at which pavement cracking occurs is higher in cold climates than in warm climates.

Figure 112 illustrates some results of their findings, and particularly the significance of weathering and temperature on strength parameters.

Table 29 contains a summary prepared by Lewis and Welborn (*82*) which associates properties of recovered asphalt and pavement performance. It is not known how the pavement evaluation was made, nor if there was any consistency in performance criteria depending on the person making the evaluation; however, it appears that the amount of cracking was the key performance factor.

It is apparent from Table 29 that the critical consistency properties of asphalt associated with cracking may vary, depending on geographical location and associated climate. Thus, an asphaltic surface containing an asphalt of 20 penetration would be susceptible to cracking in Michigan, Minnesota, Ohio, etc., whereas in Arizona or Cuba the critical value of penetration could be 10 or less. Ductility and softening-point test results showed similar trends.

Krchma (*8*) has listed a number of properties of the in-place mix considered important to the weathering of asphalt, as follows:

1. Pore surface area.
2. Permeability.
3. Film thickness.
4. Aggregate surface effects.

There seems to be general agreement that these four fac-

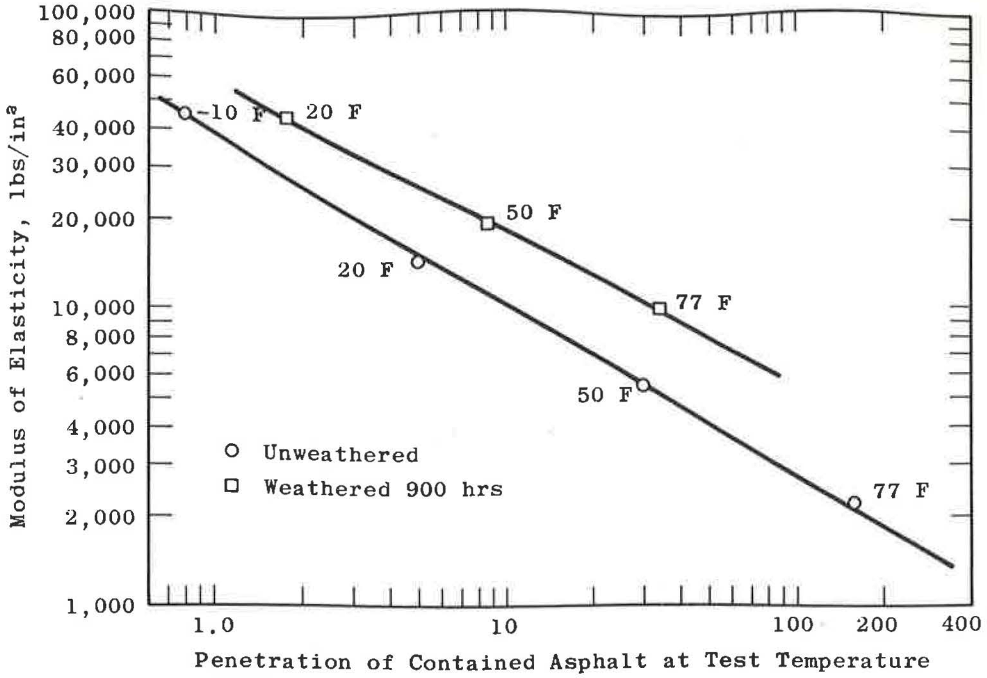


Figure 112. Relation between modulus of elasticity and penetration of contained asphalt at test temperature (after Lewis and Welborn, 83).

tors influence the rate of hardening of asphalts presently produced in the United States. Figure 113 shows, as an example, the effect of air voids (pore surface area) and emphasizes the need for obtaining good compaction and low voids. Krchma and Groening (*77*) suggest the following conclusions after an investigation of the influence of voids, asphalt content, and asphalt grade on surfacing performance:

TABLE 29

CORRELATION OF PAVEMENT CONDITION WITH PHYSICAL PROPERTIES OF RECOVERED BITUMEN [a]

DATA SOURCE	PAVEMENT LOCATIONS	PAVEMENT CONDITION	TESTS ON RECOVERED BITUMEN: PEN., 77 F	DUCT., 77°F (CM)	SOFT. POINT (°F)
Shattuck	Detroit, Mich.	Very good	20+	50+	—
		Good	20+	25+	—
		Badly cracked	20—	25—	—
Thomas	Minnesota	Good [b]	41	—	—
		Fair [b]	26	—	—
		Poor [b]	20	—	—
Hubbard and Gollomb	Ohio, Mich., N.Y., Ind., D. Col.	Sound	30+	—	—
		Prone to crack	30—	—	—
		Cracking type	20—	—	—
Vokac	Ohio, Pa., Md., Va., Mo., Ill., Ind., Mich., N.Y., N.J., D. Col.	Sound	25+	24+	—
		Prone to crack	18–25	4–24	160—
		Cracking type	18—	4—	160+
Powers	Arizona	Good	10+	10+	160—
		Cracked	10—	10—	160+
Pub. Rds. Admin.	Cuba	Good [b]	9	1.5	199
		Cracked [b]	5	0.5	217

[a] After Lewis and Welborn (*82*).
[b] Average values.

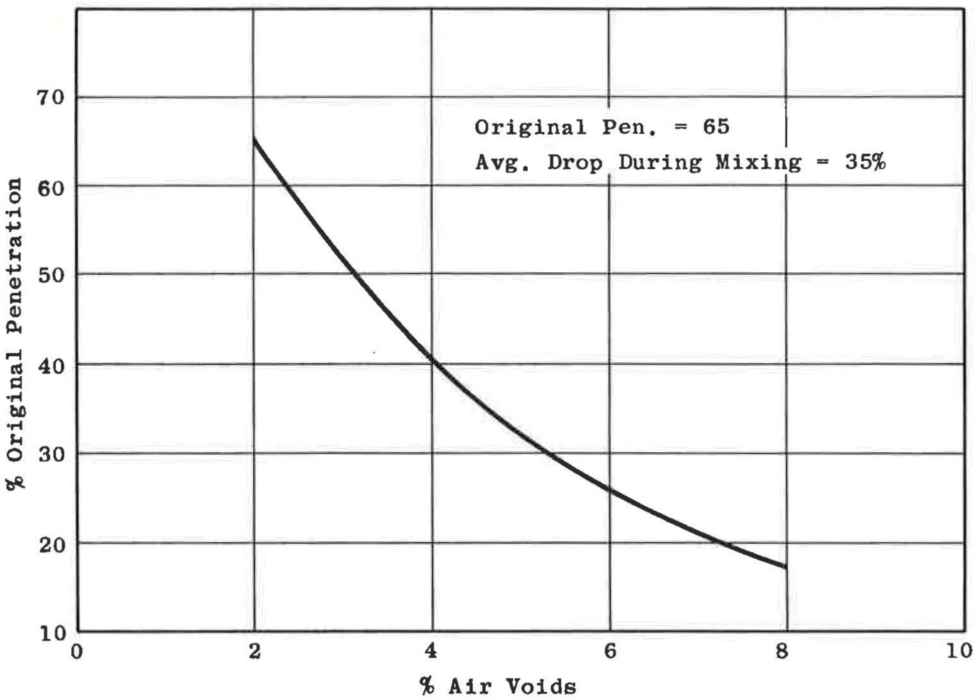


Figure 113. Voids vs rate of hardening for a Mexican bitumen after 15 years of service (adapted after Pfeiffer, 122).

1. Asphalt content must be adjusted with the asphalt grade.
2. Higher asphalt contents provide longer service.
3. Higher asphalt contents with harder grades offer an opportunity to obtain better surfacing durability.

Commenting on the significance of using initially hard asphalts, Krchma indicated that in England heavy-duty roads are built using asphalt having an initial penetration of 40, and country roads are built using asphalts with an initial penetration of 60 to 70. He stressed the point that when using asphalts of low initial penetration, it is necessary to have relatively thick films for durability. However, no specific values of film thickness were suggested. The main point made by Krchma was to emphasize that other factors in addition to the properties of the asphalt can influence the durability of a mix. Significantly, most of these other factors can be modified or controlled by the engineer (e.g., adjustment of gradation, compaction, asphalt grade, and asphalt content).

Clark (*8*) has described asphaltic surfacing durability in terms of asphalt ductility. Apparently, he was able to develop an accelerated weather test in which a week of aging was representative of a year of actual service. The correlation between field and laboratory aging was based on the results of ductility tests on recovered asphalts. It is interesting to note that Clark used samples of asphaltic concrete to produce accelerated asphalt weathering. He tested 48 projects, 31 of which have some available field performance. Of these, cracking occurred on 12 sections. Although the exact cause of cracking may not always be related to asphalt, it seems highly significant that after three weeks of accelerated weathering those sections with cracking had ductility values of 15 cm or less. Of those 19 sections without cracking, only one had a ductility less than 15 cm. Thus, Clark concludes that ductility is a good measure of asphaltic surfacing durability.

Doyle (*30*) described two new tests—(a) brittle point test, and (b) linear thermal expansion test—for asphalt cement. Based on laboratory research, these tests were believed to identify the necessary properties of a durable asphalt. The brittle point test was designed to define the lowest temperature at which an asphalt cement specimen could sustain a given deformation without fracture. The linear expansion test was developed to measure the volumetric change of an asphalt cement subjected to temperature change. The identifying property of this test could be expressed as (a) the temperature at which the asphalt changes from the solid to the viscous state, as noted by a transition in the slope of the volume change curve, or (b) the slope of the expansion curve below this transition temperature.

On the basis of the laboratory tests, Doyle concluded that asphalts should have a brittle point of approximately 0 F coupled with the lowest possible linear thermal expansion coefficient.

A particularly noteworthy aspect of this investigation was the fact that Doyle was able to have asphalts with a low brittle point produced in sufficient quantity for use in two test roads. Thus, he was able to evaluate the practical significance of his laboratory research. Linear thermal expansion coefficients for these special asphalts were reported as being unchanged from normal production. For comparison, the test roads included control sections made with regular asphalts used by the Ohio State Highway Department. The primary dependent variable for asphalt evaluation was the tendency of the surfacing to crack after being combined with aggregate and placed on the road.

Some test results obtained from this study and considered pertinent to durability are summarized in Table 30. Doyle reported that after two years of service under heavy loads the surfacing with synthetic asphalts showed noticeable cracking, and after three years developed additional cracking. No cracking was reported on either project for surfacings made with regular asphalt. He concluded that although the special asphalts were bendable at low temperatures they ". . . lacked the characteristic of being able to slowly elongate at colder temperatures." For example, asphalts recovered from cracked surfacing had ductilities of 1 and 6 cm at 55 F, whereas asphalts recovered from uncracked surfacing had corresponding ductilities of 11 and 12 cm. Similar comparisons of the original asphalt and the oven-weathered asphalts are also given in Table 30. It is interesting to note that asphalts A and B, the synthetic asphalts, with retained penetrations of 60 and 78, respectively, after the BPR thin-film oven test, showed early distress in the test roads.

Although data from the two test roads appear to be inconclusive as regards the brittle point, Doyle continued to investigate ductility at 55 F. He suggested, as the result of further field correlations, that two tests could be used to predict susceptibility of surface cracking as a function of asphalt properties; namely, (a) ductility on original asphalt at 55 F, 1 cm/min, and (b) ductility of recovered asphalt at 55 F, 1 cm/min, after 1-hr oven weathering. The oven temperature was equal to the temperature required to produce a 120-sec Saybolt Furol consistency.

In a more recent paper, Doyle (*31*) points out that although asphalt penetration (original, weathered, or recovered) has been used for more than 40 years to measure asphalt hardening, the test can no longer be depended on, in some sections of the country, due to the crossblending of crudes and due to the use of additives and inhibitors. On the basis of the two test roads previously described, together with field observations and laboratory correlations, Doyle (*30*) suggests that low-temperature ductility is a good indicator of a durable asphalt for an asphaltic surfacing. To obtain better definition between "good" and "bad" asphalts, he suggests (*31*) performing the ductility test at 45 F, 1 cm/min, for both original and recovered asphalt cements. He reports that any paving mixture containing an asphalt with a ductility (45 F, 1 cm/min) greater than 8 cm should be free of cracking. To have a safe working value, he recommends a minimum ductility of 25 cm. He also described a procedure for simulating hot-mix hardening to provide estimates of long-term asphalt durability.

Doyle has stressed the point (*30, 31*) that an unnecessary loss of ductility may occur in an asphalt if it is overheated during mixing with the aggregate. To illustrate this point, he simulated plant mixing operations using the recommended asphalt temperature corresponding to 120-sec Saybolt-Furol and, for comparison, mixing temperatures of 325 F. Table 31 illustrates the effect of this difference in temperature on the ductility of the recovered asphalt measured at 45 F, 1 cm/min. If properly mixed, only the 50-60 asphalt would be critical. If overheated (325 F) during mixing, only the 85-100 asphalt would be expected to provide satisfactory performance. These data tend to indicate that it would be difficult to meet the ductility requirement of 25 cm previously suggested.

Based on his research, Doyle suggested that asphaltic concrete, for asphaltic surfacings, be compacted initially to 5½ percent air voids.

Halstead (*44*) presents a case for considering the advantages of using the ductility-penetration relationship of an asphalt as a measurement of its contribution to pavement performance. An extensive analysis of available information permitted him to develop a unique relationship between penetration and ductility of asphalts from different crude

TABLE 30

ASPHALT TEST RESULTS [a]

ASPHALT IDENT.	ORIG. PEN. AT 77 F	DUCTILITY ON ORIG. ASPHALT: AT 77 F, 5 CM/MIN	DUCTILITY ON ORIG. ASPHALT: AT 55 F, 1 CM/MIN	DUCT. ON RECOVERED ASPHALT, AT 55 F, 1 CM/MIN [b]	PEN. AFTER BPR THIN FILM, AT 77 F	DUCT. AFTER MIXING OVEN WEATHERING TEST, AT 55 F, 1 CM/MIN	BRITTLE POINT, SOHIO (°F)	LINEAR COEFF. (IN./IN./°F)
Test Road No. 1:								
B [c]	108	22	6	1	78	4	0	11.5×10^{-5}
C [d]	108	110	101	11	67	17	22.5	6.1×10^{-5}
D [d]	76	100+	—	—	49	—	32.5	7.06×10^{-5}
Test Road No. 2:								
A [c]	94	110+	15	6	60	7	13.5	—
E [d]	88	110+	102	12	61	19	33	—

[a] Adapted from Doyle (*30*).
[b] After 3 years.
[c] Synthetic asphalt, special production to provide a low brittle-point temperature.
[d] Regular asphalt.

TABLE 31

LOSS OF DUCTILITY AS A FUNCTION OF MIXING TEMPERATURE [a]

ASPHALT MIXING CONDITION	PENETRATION DESIGNATION (DMM)	DUCTILITY, AT 45F, 1 CM/MIN
Temp. equal to 120-sec Saybolt-Furol	85–100	19
	70–85	13
	60–70	8¼
	50–60	7½
325 F	85–100	10½
	70–85	6½
	60–70	5¼
	50–60	5

[a] Adapted after Doyle (*31*).

sources and their field performances. On the basis of the information presented, an effort was made to establish a "critical" ductility-penetration relationship. The lower extreme for critical values would be a ductility of 10 cm and penetration of 25. The upper extreme would be a ductility of 100 cm and a penetration of 50. Halstead indicates that the combination of ductility and penetration will significantly improve the relationship of these physical constants to performance. He also indicates that the physical characteristics of the pavement, such as void content, permeability, and environmental factors, greatly affect the hardening rate of the asphalt.

The research described in the preceding portion of this chapter is intended only as a representative sampling or cross section of research which has been reported in the literature. For the most part, these investigations, and many others not specifically cited (*10, 56*), have generally attempted to relate some type of consistency test results to performance. These tests will sometimes, but not in every instance, include some form of accelerated weathering to obtain the long-term effects of in-service aging.

The net result of these investigations rather definitely indicates that the harder asphalts (i.e., low penetration, low ductility, or high softening point) tend to have the poorest performance record. In some instances critical values are suggested for each test. Without exception, this type of research has shown that all asphalts tend to change in characteristics with time, always moving toward the suggested critical levels. It is pertinent at this point to question, at least in passing, the method by which the asphalt was rated. Performance was generally described in terms of cracking. For the most part, no specific information was given as to the structural designs involved. In view of information included herein relative to fatigue, it would be helpful in the future to develop specific criteria which can be used to judge the performance of the asphalt *per se*. It would be suspected that the major share of the surfacings used to evaluate asphalt were of thicknesses less than 4 in. If this was the case, the "harder" asphalts would be expected to have a higher tendency for fatigue cracking.

It must be emphasized that the research efforts previously referred to in this chapter include only a small fraction of those published on the subject of asphalt durability as measured by changes in asphalt consistency. It is believed, however, that this selected sampling is representative of the conclusions expressed by the majority of investigators on this subject.

In recent years a considerable effort has been made to study asphalt on the basis of its chemical composition (*38, 43, 128, 129, 130, 135, 136, 152*). This is a more fundamental approach to understanding asphalt and its rheological and weathering properties. Some have suggested that the greatest potential for such techniques is in the manufacture of asphalts to produce specified physical properties. A detailed discussion of these research efforts is beyond the scope of this report. However, in view of the possible effect this work may have on future asphalt specifications or asphalt manufacturing procedures, a brief review is included.

Schweyer et al. (*135*) and Rostler and White (*128*) suggest that the chemical approach to asphalt technology should yield the type of fundamental information needed to produce optimum engineering properties. The consensus expressed by these investigators indicates that durability and adhesion are the most likely properties to be controlled by composition, but that eventually rheological properties should also be definable by such methods.

Rostler and White (*129*) have published information illustrating the relationship between asphalt durability and chemical composition as determined by methods developed by the authors. In their investigations durability was measured by means of a pellet abrasion test which was developed to correlate with California's shot abrasion test. This test is designed to evaluate the ability of the asphalt to cement together coated particles of Ottawa sand. Chemical composition is expressed as the ratio of the reactive (nitrogen and first acidaffins) to the less reactive chemical compounds (second acidaffins and paraffins) in the asphalt. Table 32 describes the characteristics of these fractions in terms of their function as part of the asphalt. Rostler and White (*129*) were able to divide 85-100 penetration asphalts into nine durability groups from this first effort. Rostler (*130*) later suggested that the number of groups could be reduced to five.

Of particular significance from this research is the demonstrated independence of asphalt properties of a particular geographical origin of crude oil to durability. Rostler and White (*129*) were able to blend fractions obtained from crude sources giving low asphalt durability into an asphalt of the highest durability. They were also able to blend fractions from many sources to produce a predetermined durability rating.

Halstead, Rostler, and White (*45*) present additional information relative to chemical composition and durability, in terms of the pellet abrasion test, with a wide range of asphalts. This effort continues to support the feasibility of (a) durability grouping of asphalts by chemical composition and (b) the introduction of asphalt viscosity as an additional parameter to assist in predicting the cementing properties of asphalt.

TABLE 32

CHARACTERISTICS OF ASPHALT COMPONENTS[a]

FRACTION DESIGNATION	DESCRIPTION	SPECIFIC CHEMICAL REACTIVITY	ELEMENTARY COMPOSITION (%) C	H	N	S	O	SIGNIFICANT FUNCTION
A, asphaltenes	Higher molecular weight condensation products	Insoluble in *n*-pentane	86.5	8.3	2.3	1.5	1.4	Bodying agent
N, nitrogen bases	Petroleum resins containing nitrogen bases and other highly reactive compounds	Precipitable with 85% H_2SO_4	86.1	9.3	3.0	0.7	0.9	Peptizer for asphaltenes
A_1, first acidaffins	Resinous hydrocarbons	Precipitable with conc. H_2SO_4	90.0	7.8	0.3	1.9	—	Solvent for peptized asphaltenes
A_2, second acidaffins	Slightly unsaturated hydrocarbons	Precipitable with fuming[b] H_2SO_4	88.8	9.6	0.0	0.9	0.7	Solvent for peptized asphaltenes
P, paraffins	Saturated hydrocarbons	Non-reactive with fuming[b] H_2SO_4	86.5	13.5	—	—	—	Geling agent

[a] Adapted after Rostler and White (*129*).
[b] 30 percent SO_3.

AGGREGATE DURABILITY

A relatively small amount of information is available in the literature concerning the role of aggregate durability in the performance of asphaltic surfacings. Considering that approximately 85 percent of the total volume of the asphaltic mixture is composed of aggregate, this is somewhat surprising. However, it seems reasonable to conclude that this lack of research must imply that present test requirements for aggregate hardness or aggregate durability are adequate to meet the demands of time and traffic.

One of the principal forms of inadequate durability of aggregate in asphaltic surfacings is aggregate degradation. Hveem et al. (*67*), Minor (*6*), and White (*166*) described this phenomenon for West Coast aggregates. For the most part, it does not appear to be a problem in asphaltic concrete. Minor does cite instances of failure of asphaltic concrete due to poor mineral aggregates. He indicates that had stripping tests or immersion compression tests been routinely carried out at the time, the difficulty would have been detected in the materials evaluation stage. Moavenzadeh and Goetz (*93*) report results of a laboratory study to evaluate degradation of aggregate in bituminous mixtures. The magnitude of degradation was based on the increase in surface area produced by subjecting specimens to the action of a gyratory testing machine. The results of this investigation are summarized as follows:

1. The magnitude of degradation of aggregate in a bituminous mixture depends on the kind of aggregate, gradation, compactive effort, and particle shape.
2. The Los Angeles (rattler) test reflects the tendency for degradation of aggregates in bituminous mixes.
3. Some aggregates with a high Los Angeles loss, greater than commonly specified, may be satisfactory if used in dense gradings with low compactive effort.
4. Gradation of the mixture is the most important factor controlling degradation, the denser gradations resulting in less degradation.
5. Increasing compactive effort, either in terms of repetitions or load, increases tendency for degradation; however, increasing load has the greater effect.
6. Rounded particles have less tendency for degradation than angular particles of the same kind of aggregate.

In addition to aggregate durability, *per se*, there is also the factor of aggregate-asphalt adhesion, particularly in the presence of water. This factor is included herein with the discussion of aggregate durability.

Ref. (*5*) contains a series of papers concerned with the effect of water on bituminous paving mixtures. For example, Parr (*5*) summarized field observations reported to him from broad geographical areas of the United States on the detrimental effects of water on asphaltic surfacing. In summarizing his information for hot-mixed asphaltic pavements, he states: ". . . it is clear that dense-graded pavements properly designed for gradation and bitumen content and compacted during construction to the required density apparently suffer no adverse effect upon exposure to moisture." He points out, however, that this statement in effect imposes very strict requirements for design and construction. For example, poorly graded aggregates which produce open grading or which contain an excess of poor quality fines will develop poor adhesion, which can result in stripping of the asphalt from the aggregate. Inadequate asphalt content, lack of compaction, or inadequate drying of the aggregate can also lead to stripping.

The following general conclusions appear warranted, based on papers (*5*) by (a) Rice, on mineralogical and chemical composition of aggregate with reference to asphalt adhesion and chemical reactivity; (b) Klinger and Roediger, on laboratory and field studies to evaluate stripping; (c) Critz, on the effectiveness of anti-stripping additives evaluated by various laboratory measuring procedures; (d) Brown, Sparks, and Marsh, on the study of asphalt stripping from aggregates; and (e) Goetz, on methods of testing water resistance of bituminous mixtures:

1. Although there are theoretical reasons why an asphalt-aggregate system might be undesirably affected by water as regards adhesion, no extensive problems have been reported .n dense-graded, well-compacted mixtures.
2. Anti-strip additives generally improve the immersion-compression test results (depending on the heat stability of the additive) and reduce stripping in the static immersion test.
3. A major difficulty in evaluating aggregate-asphalt stripping or adhesion is the lack of a definitive laboratory test that does not depend on visual estimates of test results.

Zube and Cechetini (*169*) have reported on non-load-associated cracking resulting from the use of absorptive aggregate. Their studies were concerned with volume change phenomena associated with aggregates of high absorption. It was found that mixes made with absorptive aggregate and exposed to normal atmospheric conditions exhibit daily expansion and contraction cycles, with the net cumulative effect being one of expansion. The mixes also exhibited visible amounts of cracking due to these cyclic volume changes. Although the use of selected fillers tended to reduce the volume change characteristics, an increase in asphalt content sufficient to increase the film thickness by 1 micron was also effective.

ASPHALTIC CONCRETE DURABILITY

It is difficult to discuss the durability of asphaltic surfacings in terms of the durability of individual components without constantly being aware of the various interactions involved. For example, asphalt durability and aggregate durability have been discussed in the previous sections from past research designed to isolate the performance characteristics of these materials. However, in isolating asphalt durability, particularly when in-service correlations are made, it is necessary to be aware of the environment to which the asphalt is exposed (e.g., surface potential of aggregate, void content of compacted mix, asphalt film thickness, layer thickness, climate, traffic, etc. Similar interactions are at play in studying the durability of aggregates.

This section deals with the durability of the asphaltic mixture, either in simulated laboratory experiments designed to accelerate aging or in actual field trials. The principal objective of this effort by the various investigators has been to define those factors in an asphalt-aggregate mixture which contribute to the life expectancy of an asphaltic surfacing.

In the previous section, the problems of asphalt-aggregate adhesion in the presence of water were discussed, with emphasis placed on the aggregate phase. Recently, Skog and Zube (*147*) have suggested the following four tests which objectively evaluate adhesion of the mixture:

1. Quantitative dye stripping test.
2. Moisture vapor susceptibility test.
3. Water susceptibility test.
4. Surface water abrasion test.

The dye stripping test involves measurement of the relative amount of saframine dye adsorbed on asphalt-aggregate mixtures after being subjected to a flushing action with a solution containing the dye. One of the unique features of this test is its inclusion of the smaller size fractions, through the No. 100 sieve. Zube and Skog have concluded that it is difficult to relate laboratory stripping test results (such as the dye test) to the prediction of in-service performance. They summarize as follows: "Even though (stripping) test results do not indicate such a condition, pavement failure may be caused by a combination of stripping and partial emulsification. Conversely, an aggregate which shows serious stripping in an uncompacted state may prove satisfactory when used in a properly compacted pavement." Therefore, they suggest some type of physical test on the asphaltic concrete itself.

The moisture vapor susceptibility test is performed on the asphaltic concrete and is an attempt to simulate the adverse effects of moisture vapor on the physical strength of the mix. In the test, the specimen is exposed to a humid, almost saturated, atmosphere for a specific amount of time, after which a stability test is performed. The retained stability is a measure of the resistance of the mixture to this type of environment. The MVS (moisture vapor susceptibility) test has been used by the California Division of Highways and is considered a reliable test, particularly for identifying poor quality fines in the mineral aggregate.

The water susceptibility test is still in the research stage, although it is proposed as a possible routine test. This test forces water of various mineral contents through an asphaltic concrete briquette. Evaluation of the effect of the water on the specimen is based on measurements of stability, cohesion and swell.

The surface water abrasion test is an attempt to measure the potential raveling of a well-designed, well-compacted, asphaltic concrete. In this method the surface of a 4-in. diameter asphaltic concrete specimen is subjected to the abrasive action of four 1⅛-in. hard-rubber balls in the presence of water. The agitation is obtained by means of a shaker operating at 1,200 cycles per minute with a vertical stroke of 1 in. The test is performed at 100 F for 15 min. Loss of weight is used as the measure of potential raveling. Figures 114 and 115 show some of the abrasion loss characteristics as a function of aggregate source, hardness, amount of binder, density, and presence of additives. The authors have suggested that a maximum limit for abrasion loss should be 15 gm. The consensus of this information is that the abrasion loss is reduced by (a) increasing the asphalt content, (b) using the lower penetration asphalts, (c) increasing the density (as indicated by pressure in Fig. 115), (d) using selected additives, and (e) using selected aggregates. Aggregate source is indicated to be the most important single factor in the performance of the mixes tested and can override all of the other efforts to achieve a mix giving satisfactory performance.

Goode and Lufsey (*41*) have reported on a recent investigation associated with mix durability. These investigators have demonstrated, under laboratory conditions, the effect of air voids, air permeability, and bitumen index * on changes in Marshall stability and retained penetration of asphalt.

Figures 116, 117 and 118 summarize part of the results reported by Goode and Lufsey for three aggregate gradations, as follows:

SIEVE SIZE	PERCENT PASSING		
	A	C	E
½ in.	100	100	100
No. 4	73	64	56
No. 30	31	24	18
No. 100	16	13	10
No. 200	12.3	9.5	7.5

The following conclusions were reached:

1. Gain in Marshall stability is inversely proportional to the retained penetration.
2. Increasing air voids result in a reduction of the retained penetration as a function of the gradation.
3. Increasing air permeability results in a reduction in retained penetration as a function of gradation.
4. Increasing the bitumen index results in an increase in the retained penetration as a function of gradation.

$$\text{* Bitumen index} = \frac{\text{Percent asphalt (aggregate basis)}}{100 \times \text{Surface area (sq ft per lb of aggregate)}}$$

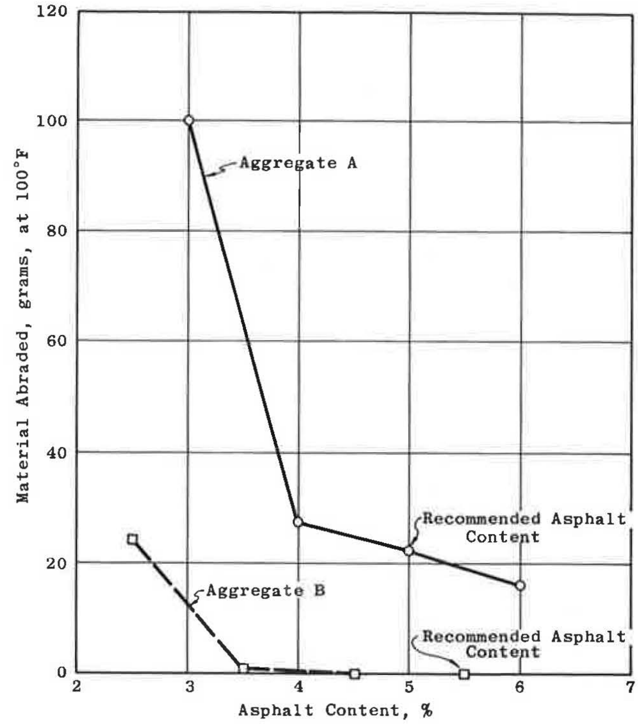


Figure 114. Effect of asphalt content on abrasion loss (after Skog and Zube (147) *by permission).*

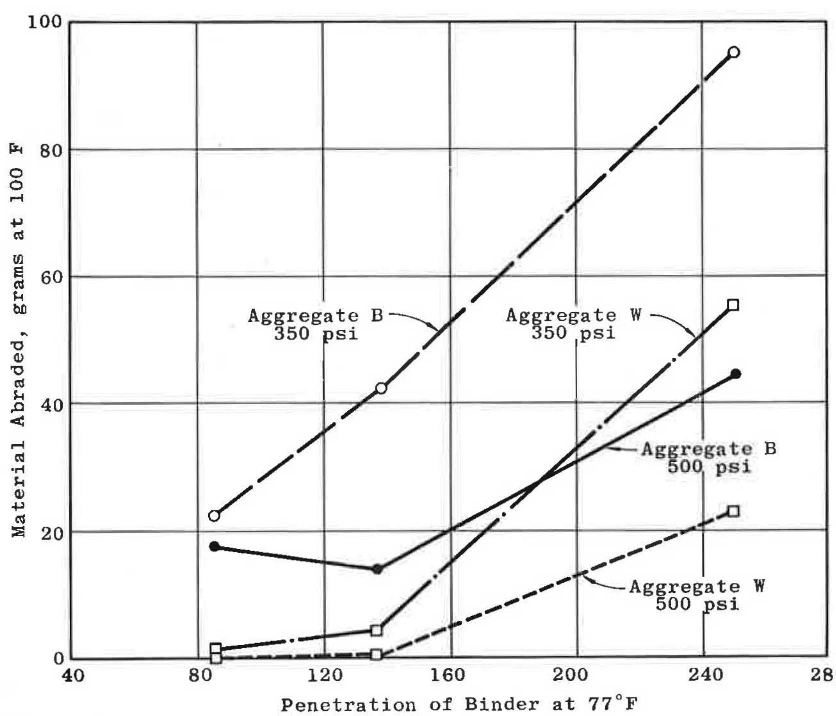


Figure 115. Effect of laboratory compaction pressure on abrasion loss (after Skog and Zube (147) *by permission).*

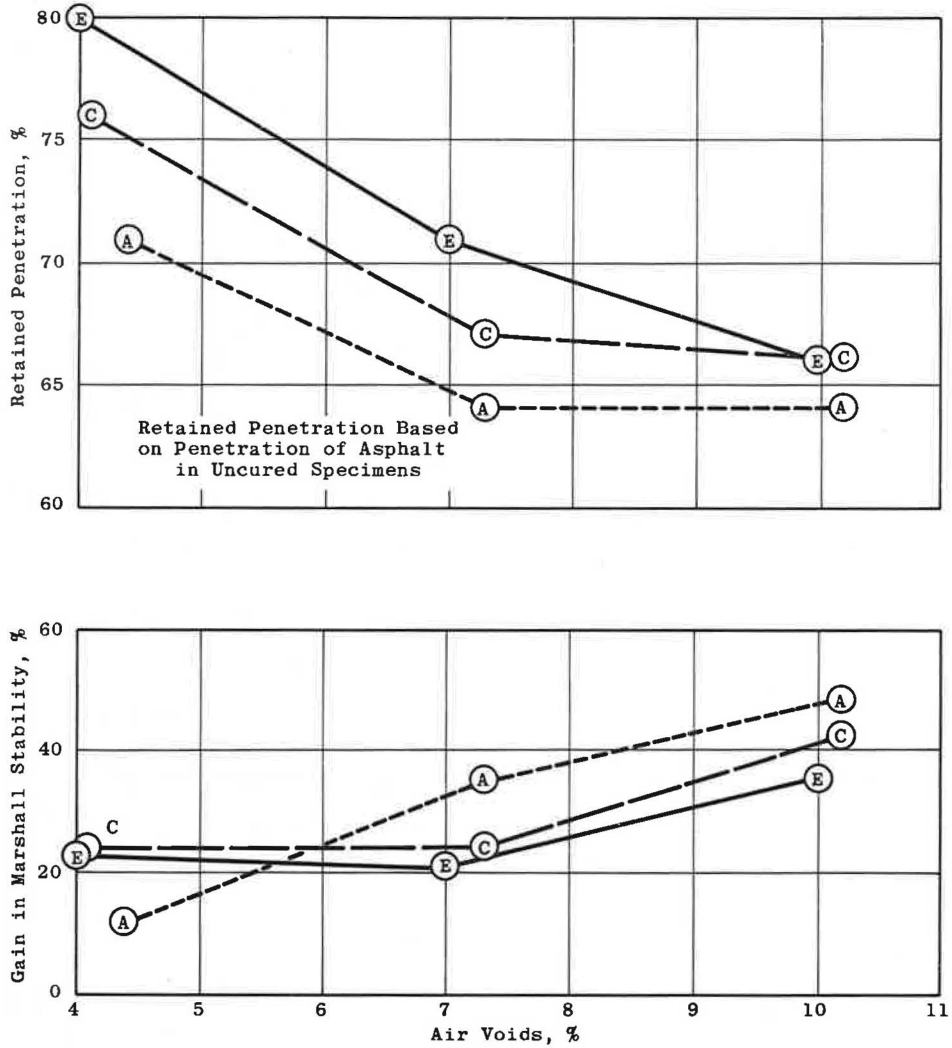


Figure 116. Effect of air voids on degree of asphalt hardening after 12-day oven curing at 140 F (after Goode and Lufsey (41) by permission).

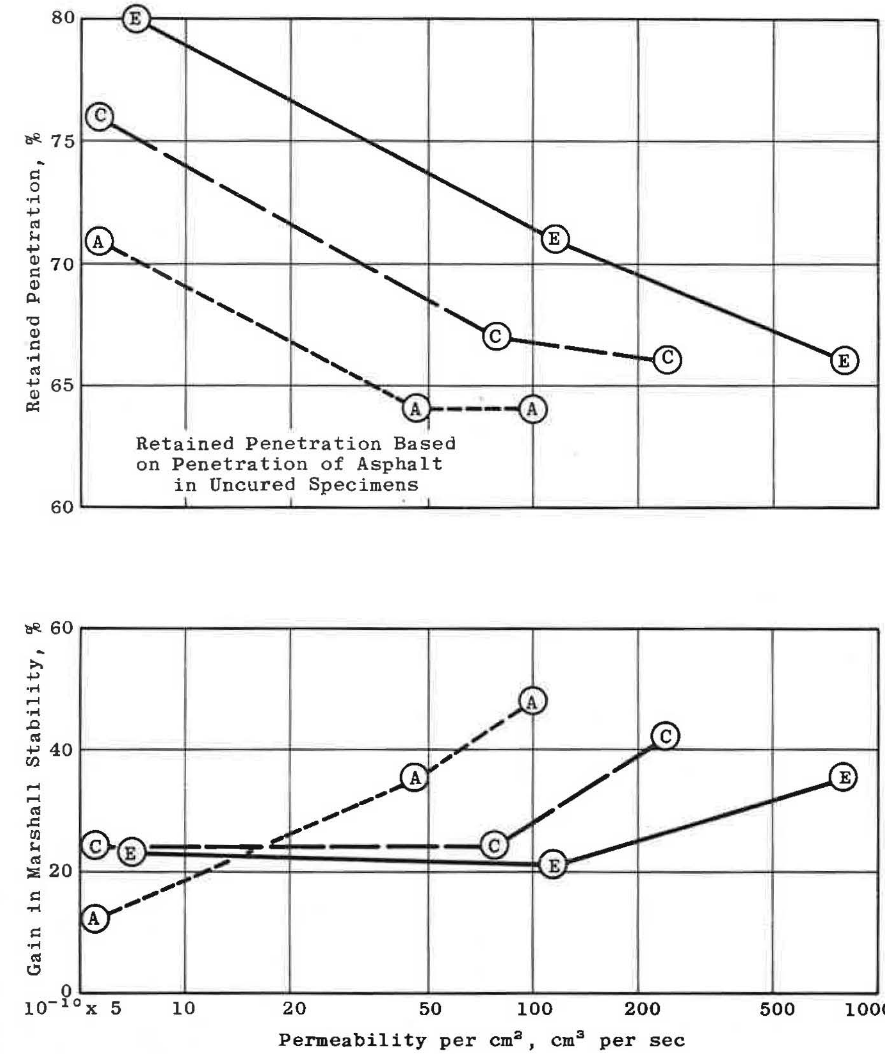


Figure 117. Effect of permeability on degree of asphalt hardening after 12-day oven curing at 140 F (after Goode and Lufsey (41) by permission).

They concluded that air voids, air permeability, or bitumen index could not eliminate the effect of aggregate gradation on asphalt hardening or mix stability (Marshall).

The ratio of air voids to bitumen index did appear to eliminate the effect of aggregate gradation and thereby offered a way to compare differences in aging independent of gradation. Another significant conclusion reported in this paper was that mixes designed to have 4 or 5 percent air voids, when compacted in the Marshall procedure, will have low air permeabilities, regardless of gradation. Thus, permeability should not be a problem at these void contents.

The following portion of this chapter describes extensive field test programs having as one of their major purposes the evaluation of the performance or durability of in-service asphaltic surfacings. Where results are available, there appears to be some consensus that the durability of the asphalt is a primary contributor to the deterioration of the surfacing; however, there is also some expression in these publications that construction can play a major role in influencing the durability of the surfacing.

Morgan (*105*) describes the behavior of special test sections constructed in North Carolina to evaluate typical aggregates from that State and to help establish laboratory design and field control procedures. Eight 1,000-ft and one 500-ft test sections were constructed. Except for the one 500-ft subsection of sand-asphalt containing 60-70 penetration asphalt cement, the surfacing in each was manufactured using an 85-100 penetration asphalt. The well-graded aggregates included crushed granite, crushed gravel, and uncrushed gravel. His conclusions, which were based on visual observations and surface roughness measurements for a three-year period, were as follows:

1. Durability and stability of bituminous wearing surfaces are directly related to densities obtained during construction.
2. No direct relationship was noted between the occurrence of stripping and mix instability.
3. No direct relationship was developed between type of coarse aggregate (stone or gravel) and pavement performance.

It is interesting to note that the test sections were constructed over an old portland cement concrete pavement and that the sections with only minor crack reflection were those in which an asphalt surface seal had been placed on the old pavement prior to placing the hot-mix. All sections without a seal developed extensive reflection cracking.

Shupe and Taylor (*144*) have reported on a study to investigate the behavior of two asphaltic concrete test sections subjected to in-service weathering and traffic over a three-year period. The purpose of the investigation was to (a) develop a better correlation between laboratory tests and actual pavement performance, and (b) evaluate the effect of anti-stripping agents. The asphaltic mixes con-

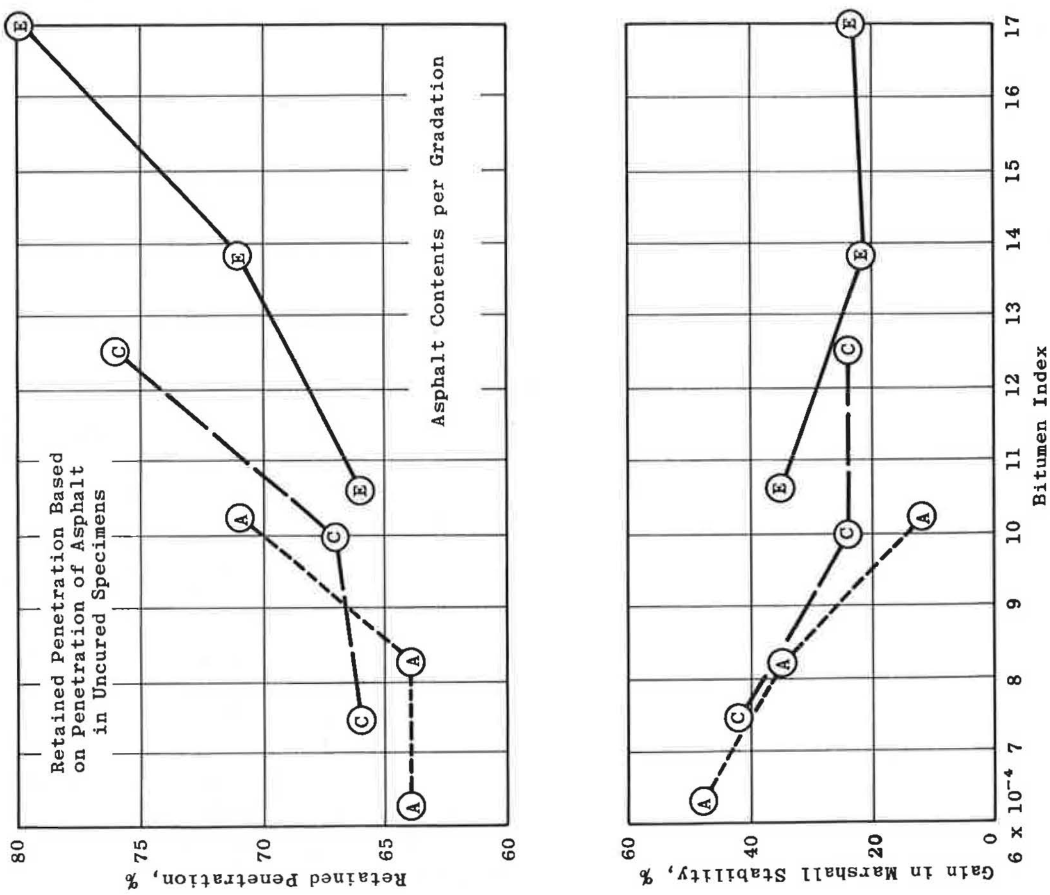


Figure 118. Effect of bitumen index on degree of asphalt hardening after 12-day oven curing at 140 F (after Goode and Lufsey (41) by permission).

tained both limestone and chat aggregates with 7 percent 85-100 penetration asphalt. Tests were performed on field samples removed from the pavement after final rolling and on laboratory specimens prepared with samples of the mixture obtained during construction. Laboratory tests included specific gravity, Marshall stability, immersion-compression, and sonic modulus of elasticity. After two years of service there were no observable indications of surface deterioration. After the third year the surfacing with the chat aggregate (Newton section) did show some loss of the coarse size fraction, particularly in the areas without additives. However, there were no signs of wear in the surfacing with the limestone aggregate (Ogden section). In this regard, it would be impossible to compare the two aggregates because traffic on the Newton section was approximately twice that on the Ogden section. In view of the observed performance of this surfacing and the reported properties of the asphalt, it is pertinent to review this project. Table 33 summarizes test results on the asphalt cement. It is difficult to evaluate the effect of the additives from these test results without some knowledge of the number of tests performed and their repeatability. However, Shupe and Taylor suggest that the asphalts with the additives have higher retained penetrations and ductility values. On the basis of the reported penetration, softening point, and ductility values for the Newton section, it would be concluded that raveling could occur with the combination of values reported in Table 33. It is apparent from these tests that the asphalt used in the Newton section hardened rapidly in the first two years; however, no cracking was reported.

Shupe and Taylor reported results of periodic measurements of sonic modulus of elasticity in the two test projects. Table 34 summarizes these test results. It is interesting to note that the sonic modulus actually increased during the three years of service. This corresponds qualitatively to the increase in density occurring due to traffic. It might be concluded that little or no fatigue cracking has occurred in the mix.

Table 35 summarizes tests to evaluate resistance of the surfacing to the action of water. None of these data would indicate to the materials engineer that raveling or the loss of large aggregate would occur. In this regard, the low ductility value (Table 33) appears to be the best clue to the raveling reported on the Newton section.

Heithaus and Johnson (*48*) have presented test results obtained from a series of special test sections constructed on an entry road into the Shell Oil Company's Wood River, Ill., refinery. The test sections contained a range of asphalt types and grades in two aggregate gradations that produced a dense and an open-graded mix. The primary purpose of these test sections was to evaluate the usefulness of the microfilm durability test in correlating asphalt hardening with field performance. Inasmuch as no information was given as to the performance of the surfacing, the primary dependent variable was the viscosity ratio; i.e., the ratio of the viscosity of the asphalt recovered from the test section to the viscosity of the original asphalt. In this regard, it was interesting to note the effect of the initial void content on the change in viscosity. Figure 119 shows the reduced rate of hardening associated with the initial void content, regardless of the origin of the asphalt. This is also shown in Figure 120 in terms of the penetration test. These data tend to emphasize the importance of obtaining low void content in the in-place asphaltic concrete surfacing.

One of the most significant and complete studies of the field performance of asphaltic surfacings was the Zaca-

TABLE 33
STANDARD TESTS ON THE ASPHALT [a]

	NEWTON TEST SECTION, CHAT AGGREGATE			OGDEN TEST SECTION, LIMESTONE AGGREGATE		
ASPHALT	PEN. AT 77 F	SOFT. POINT (°F)	DUCT. AT 77 F	PEN. AT 77 F	SOFT. POINT (°F)	DUCT. AT 77 F
Original asphalt:						
No additive	92	119	100+	92	115	100+
Additive A	108	117	100+	102	113	100+
Additive B [b]	112	116	100+	104	113	100+
Extracted, 1 year:						
No additive	36	141	13	44	131	57
Additive A	52	131	50	51	129	105
Additive B	38	140	15	45	133	95
Extracted, 2 years:						
No additive	33	149	8	63	125	181
Additive A	34	143	9	63	126	176
Additive B	34	147	7	62	125	188
Extracted, 3 years:						
No additive	28	153	6	54	129	151
Additive A	32	148	7	60	126	189
Additive B	39	151	6	64	127	193

[a] After Shupe and Taylor (*144*).
[b] 1½ percent additive on the Newton test section; 1 percent additive on the Ogden test section.

TABLE 34
SONIC MODULUS OF ELASTICITY [a]

AGGREGATE	ADDITIVE	SONIC MODULUS AT 40 F (PSI × 10^6) COMPACTED IN LAB.	SAWED FROM PAVEMENT ORIG.	1 YR	2 YR	3 YR
Limestone	None	1.93	1.81	2.46	[illegible].38	2.87
	1% A	1.96	1.83	2.56	2.72	2.84
	1% B	1.91	1.83	2.33	2.45	2.65
Avg.		*1.93*	*1.82*	*2.45*	*2.58*	*2.79*
Chat	None	1.15	1.05	1.72	1.82	1.67
	1.5% A	1.13	.98	1.88	2.02	2.20
	1.5% B	1.19	1.17	1.87	2.19	2.00
Avg.		*1.16*	*1.07*	*1.82*	*2.01*	*1.96*

[a] After Shupe and Taylor (*144*).

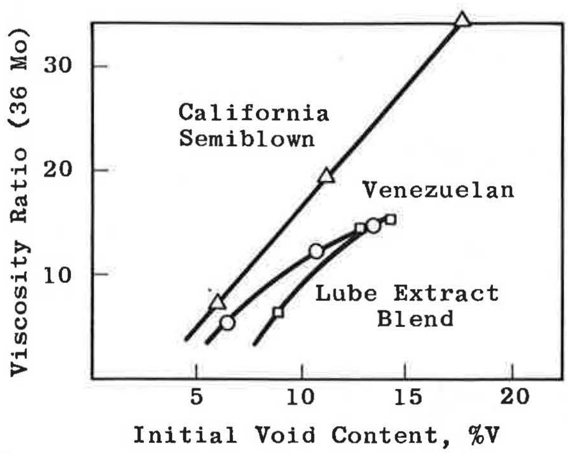


Figure 119. Effect of void content on hardening during 36 months' service (after Heithaus and Johnson (48) *by permission).*

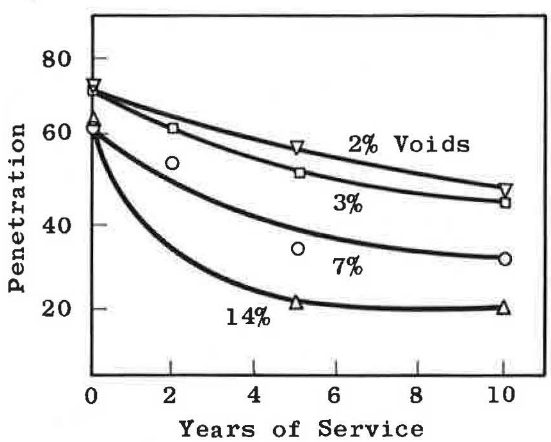


Figure 120. Asphalt hardening in several Midwestern pavements (after Heithaus and Johnson (48) *by permission).*

TABLE 35
IMMERSION-COMPRESSION TESTS [a]

AGGREGATE	ADDITIVE	STRESS (PSI) BEFORE IMMERSION	AFTER IMMERSION	PERCENT RETAINED STRENGTH
Limestone	None	757	514	68
	1% A	614	380	62
	1% B	644	455	71
Avg.		*672*	*450*	*67*
Chat	None	265	239	90
	1.5% A	295	247	84
	1.5% B	302	266	88
Avg.		*287*	*251*	*87*

[a] After Shupe and Taylor (*144*).

Wigmore Experimental Test Road, a project of the California Division of Highways. Constructed in 1954, the project had as its primary objective the evaluation of the performance of asphalts produced to meet new asphalt specifications used that year by the Division of Highways.

Hveem, Zube, and Skog (*67*) describe the work of the California Division of Highways on the Zaca-Wigmore project. The following is a summary, with particular emphasis on the performance of the surfacing and the role played by the asphalt.

The project included two types of structural sections; namely, (a) on new alignment and composed of 4 in. of asphaltic surfacing on 8 in. of cement-treated base and 12 in. of aggregate subbase; and (b) over an existing portland cement concrete pavement and composed of 4 in. of asphaltic surfacing and 6 in. of cement-treated base. Ten asphalts were included in the project, including those complying with the new (special) specifications and the standard (old) specifications. Nine of the asphalts were refined from three California crude sources and one from a mid-continent crude. The asphalts were all 200-300 penetration, the grade normally used in 1954 along the location selected for the project. Due to the construction schedule, the asphaltic surfacing was placed in two construction seasons or periods, referred to as 1, 1A (the first construction period) and 2 (the second construction period). The average asphalt content during the first period was 5.5 percent; during the second, 5.8 percent. A constant mixing temperature of 290 F was selected, regardless of the viscosity properties of the asphalt. A summary of the temperature data indicates a range of mixing temperatures from 270 to 298 F at the plant and 237 to 257 F at the paver.

Continuous sampling and testing was carried out on the project and reported by Hveem et al. Figure 121 summarizes test results for certain of the physical properties of the mix. Of particular interest is the reduced void content reported for the second paving period. Also reported were aggregate gradations as a function of time. Some degradations occurred for the surfacing placed in construction period 1 and 1A (e.g., the percent passing the No. 200 sieve increased from 4 percent to 7 percent). Practically no degradation was noted for mixes produced during paving period 2. Relative to Figure 121, the low stability values at the time of construction were associated with the low in-place density. It is significant that no instability was evident in any of the sections during the period of evaluation.

For purposes of this report, evaluation of test data and surfacing performance described by Hveem and his associates is concerned primarily with the differences which occurred between the asphaltic surfacings with acceptable and unacceptable performance. The evaluation of service performance of the various test sections was based on visual observations (ratings) and crack surveys. Table 36 summarizes the surface condition as rated by a picked rating team. According to the ratings, asphalts E, F, and G would

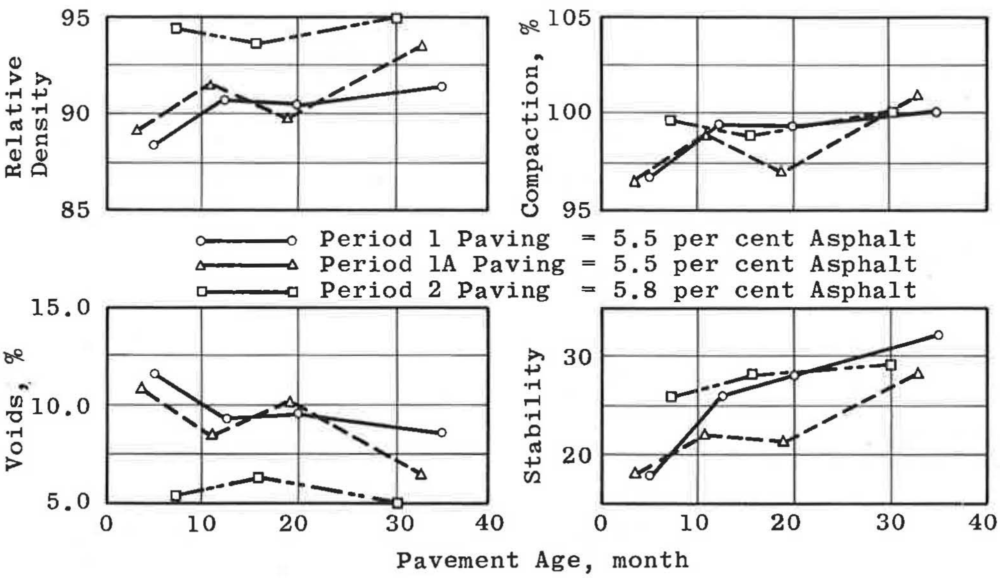


Figure 121. Average properties of pavement cores (after Hveem, Zube and Skog, 67).

TABLE 36

ASPHALT PROPERTIES ON SECTIONS OF NEW ALIGNMENT WITH LOW SERVICEABILITY RATINGS [a]

ASPHALT	RATING [b]	AGE (MONTHS)	PENETRATION (0.1 MM)	SOFTENING POINT (°F)	DUCTILITY (CM)	THIN FILM (% OF ORIG. PEN.)	ABRASION LOSS (GM)
E	Failed	0 [c]	228	99	100+	27.7	1.6
		45	10	161	5	—	182.4
F	7	0 [c]	225	102	100+	34.7	3.3
		54	25	144	[d]	—	51.9
G	5	0 [c]	242	101	100+	35.1	18.0
		54	23	151	8	—	42.7

[a] Adapted after Hveem, Zube, and Skog (67).
[b] Ranged from 1 to 10, with 1 being the highest or best performance.
[c] Original asphalt.
[d] Very erratic; average = 48, range = 7-83.

be considered poorest of those tested and particularly poor on the new alignment. Table 36 also summarizes information relative to the asphalts used on these sections after approximately 4½ years of service. By interpolation, Hveem et al. suggested that the critical test properties for asphalt E were penetration = 16, softening point = 151 F, ductility = 16 cm, and shot abrasion loss at 65 F = 128 gm.

The basic reference includes complete information on routine and special tests on all asphalts used. Figures 122 and 123 are typical of the type of analysis made in the report. Remembering that asphalt E had an unacceptable performance, both the penetration and the shot abrasion loss indicate this asphalt to have the poorest durability properties. Asphalt A was reported to have demonstrated good performance and asphalt D fair performance. Examination of the test data (Table 36) on the original asphalt indicates that the thin-film oven test results also correlate with the poor performance of asphalts E, F, and G. It is worth noting that the ductilities of these asphalts have also dropped, whereas the asphalts in the acceptable sections have retained a higher percentage of their original properties.

From the performance of the asphaltic surfacing reported, it can be noticed that those sections constructed in paving period 2 outperformed sections constructed in period 1 and 1A. There are four explanations offered for this difference: (a) the decreased amount of voids resulting from increased density and asphalt content, (b) the increased asphalt film thickness produced during period 2, (c) the lower overall deflections in the overlay sections (period 2), and (d) a general improvement in asphalt properties during paving period 2. It is difficult to assess any simple dominating cause for the difference in performance. A special section constructed in period 2 with 5.8 and 6.3 percent asphalt showed a definite improvement in the performance of the surfacing with the increased asphalt (thicker films and lower voids). Thus, it must be concluded that the use of higher asphalt contents in period 2 is a contributing factor to the improved performance. Hveem et al. conclude, on the basis of their observations and measurements, that asphalts produced from different crude sources and production can have different durability properties and consequently do influence asphaltic surfacing performance. The authors recognized that mix design and construction requirements can definitely influence the performance of the surfacing; however, they point out, and their data tend to verify the point, that durable asphalts may perform well even under unfavorable circumstances.

Parr and Serafin (*114*) have described results of a four-year test road in Michigan. The purpose of this project was to correlate asphalt properties to the service behavior of asphaltic concrete surfacing. Test variables included crude source, asphalt content, and rolling (compaction) temperature. Performance was based on field observations together with roughometer and skid tests. Parr and Serafin indicated that they believed the recovered asphalt ductility to be particularly significant as an indicator of performance. Section 6 was the only section showing any loss in ductility during the 52 months of observation. The ductility and recovered penetration data are summarized in Table 37.

On the basis of the performance of these sections at the

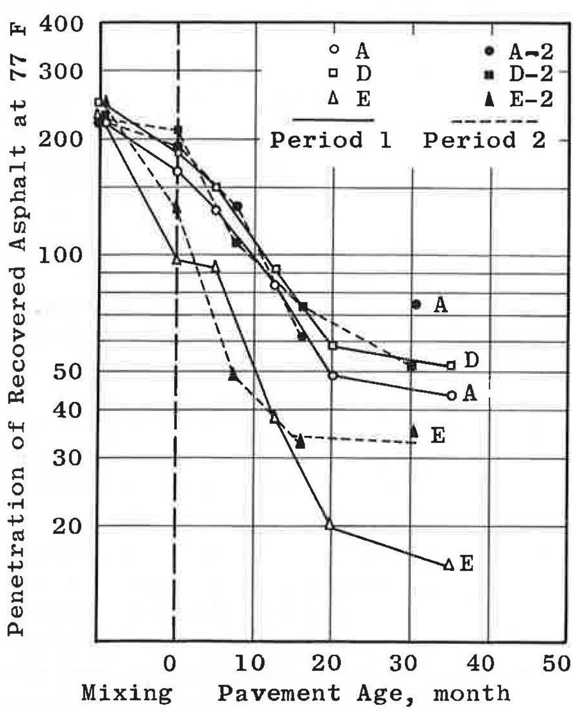


Figure 122. Change in penetration for asphalts having identical crude source and method of production for paving periods 1 and 2 (after Hveem, Zube and Skog, 67).

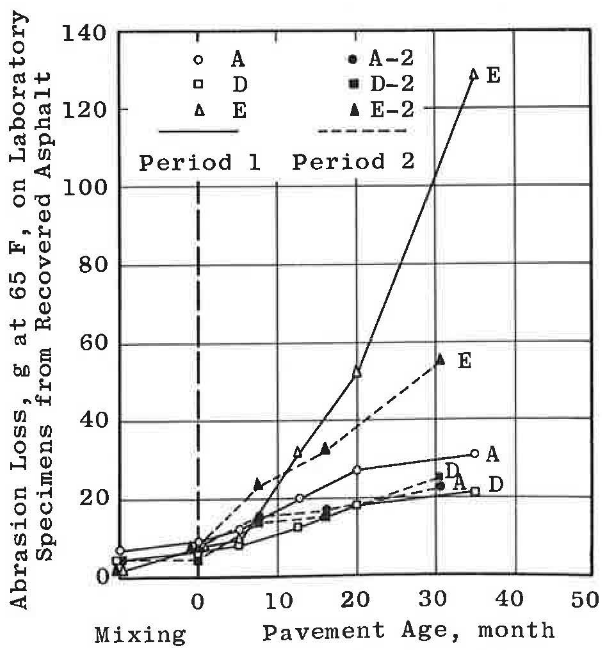


Figure 123. Change in abrasion loss for asphalts having identical crude source and method of production for paving periods 1 and 2 (after Hveem, Zube and Skog, 67).

TABLE 37

AVERAGE PENETRATION AND DUCTILITY VALUES RECOVERED FROM SECTION 6 [a]

AGE (MO)	TRAFFIC LANE		PASSING LANE	
	PEN.	DUCT.	PEN.	DUCT.
0	43	144+	45	150+
3	44	125+	41	106
17	39	104	34	59
29	44	147+	35	89
40	38	140+	35	62
52	39	138+	28	12

[a] After Parr and Serafin (*114*).

time the report was published, it was not possible to draw conclusions about the mix or its components because no significant deterioration was reported. It is interesting to note in Table 37 that the asphalt in the passing lane decreased in penetration and ductility faster than that in the traffic lane. This was concluded to be associated with the higher overall level of voids in the passing lane. It was suggested that a higher asphalt content could be used in the passing lane to compensate for the normal reduction in voids attributal to heavy traffic in the so-called "traffic" or outer lane. It is pertinent to note that all of the test sections were approaching, somewhat asymptotically, an air void content of 3 percent at the last determination reported.

White (*166*) has reported on the service behavior of asphaltic concrete for Oregon highways. The purpose of this study was to determine the effect of time and traffic on asphaltic surfacing and to compare laboratory compacted specimens of asphaltic concrete with specimens obtained from in-service pavements. The project was initiated in 1954 and included some 872 miles of asphaltic surfacing that had been in service from one to eleven years. Measures of in-service performance (i.e., cracking, raveling, shoving, flushing, and wheel track depressions) were used. For laboratory comparisons, tests of density and stability (Hveem) were used.

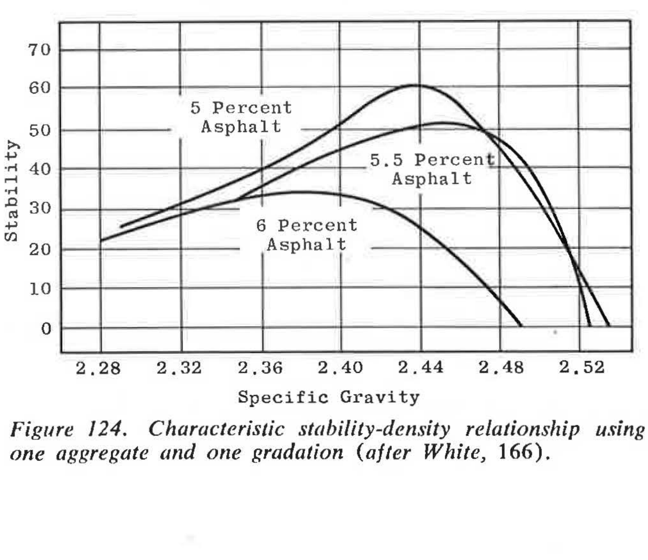


Figure 124. Characteristic stability-density relationship using one aggregate and one gradation (after White, 166).

The conclusion to be drawn from the in-service performance was that a measurable amount of cracking was occurring in more than 40 percent of sections evaluated after some six years of service. It was concluded that an increased amount of asphalt cement would have reduced the amount of cracking. No shoving was observed. A minor amount of raveling and flushing was reported. However, a considerable amount of wheel track depression was reported. On the basis of measurements and observations, White concluded that these depressions were the result of densification under traffic. Measurements of aggregate gradation, at construction and after one to eleven years of service, indicate little degradation. For example, the average increase in the percent passing the No. 200 sieve was less than 1 percent, with similar results throughout the usual range of sieve sizes. Of somewhat more interest was the apparent loss of asphalt as indicated from comparative extraction tests. For 169 specimens tested, 78 percent had less asphalt than reported from extraction tests at the time of construction. This loss could be due to absorption of the asphalt by the aggregate and would tend to produce dry appearing mixes and the possibility of some non-load-associated cracking.

Figure 124 shows the general trend of density and in-place Hveem stability found by White. For example, the initial field compaction and stability is represented by the left side of the curves. The density and stability increase with the application of traffic, approaching the peak stability values shown.

The important overall conclusion reached by White is the need to increase density requirements to improve performance by decreasing the wheel track depressions. In Oregon a requirement of 95 percent relative compaction, based on ASTM D1561-58T (kneading compactor), should, he concludes, produce a density comparable to the maximum obtained from traffic.

Kenis (*76*) has described an experimental test section in Delaware. The roadway included both conventional asphaltic (flexible) pavements and portland cement concrete pavements. The latter pavements were first covered with an asphalt seal coat and were then overlaid with 3 in. of asphaltic concrete. The top 1½ in. of the overlay contained a 60-70 penetration asphalt from two separate sources. The lower 1¾-in. layer contained a 70-80 penetration asphalt refined from a mixed crude. The report deals primarily with the top layer. Table 38 summarizes asphalt properties in the wearing surface based on tests reported by the cooperating agencies (Delaware State Highway Department, The Asphalt Institute, and the Bureau of Public Roads). It is interesting to note that the range of recovered ductility values reported by the agencies is quite large.

A considerable amount of information was reported in this paper about the mix properties, with particular emphasis being placed on stability and voids. It is interesting to note (Table 39) the relatively low Hveem stability tests reported. For the 5.5 percent asphalt content used, this value is not surprising considering that the core samples at construction were 93 percent of the laboratory maximum and after two years were 95 percent of the laboratory maximum.

TABLE 38

PROPERTIES OF ASPHALT RECOVERED FROM WEARING SURFACE [a]

		PENETRATION, 100 GM, 5 SEC, AT 70 F		SOFTENING POINT (°F)		DUCTILITY, 77F, 5 CM/MIN	
ASPHALT	TIME	AVG.	RANGE	AVG.	RANGE	AVG.	RANGE
A	At construction	34	32–40	136	133–139	—	—
	After 2 years	27	19–33	144	141–146	46	42–73
B	At construction	36	31–40	133	132–139	—	—
	After 2 years	26	25–28	145	141–145	45	31–63

[a] Adapted after Kenis (*76*).

TABLE 39

SUMMARY OF STABILITY AND VOID CONTENT OF CORES FROM ASPHALTIC SURFACE [a]

TIME	MARSHALL STABILITY (LB)	FLOW (0.01 IN.)	AIR VOIDS (%)	HVEEM STAB.	AIR VOIDS (%)
At construction	1190	16	7.8	21	8.2
After 2 years	1750	15	6.5	24	7.1

[a] Adapted after Kenis (*76*).

For the physical properties reported and two years of service, at an annual ADT of approximately 1,050 vehicles, no deterioration other than minor reflection cracking was reported. Kenis draws an interesting conclusion to this project, as follows:

"At present no specific conclusion can be drawn, but the laboratory tests indicate that variances in behavior of the same asphalt at different locations in the road may be as great or greater than variances in the asphalts from the different crude sources used in this study."

Gotolski, Ciesielski and Heagy (*42*) have reported on one of the more recent studies relating to asphalt properties of in-service pavements in Pennsylvania. Although these pavements have not been in service long enough to reach any conclusions, the study is included here to complete this investigation resume, and to relate some of the factors considered important by the investigators. The investigators were primarily interested in the durability of the asphaltic surfacing, but believed that asphalt was one of the prime contributors to this property and, therefore, concentrated on this factor.

Four projects were included in the study. The projects were constructed in 1961 and 1962, using similar construction procedures on each. The roads were asphaltic concrete overlays using 70-85 or 85-100 penetration asphalt. It is interesting to note that for these projects the average retained asphalt penetration, after mixing, was 69 percent with a range of 56 to 91. Similar information reported from the Zaca-Wigmore project by Hveem et al. indicates that the average value was 71 percent for those asphaltic surfacings which demonstrated acceptable service.

Gotolski et al. included in their program a series of special tests, including (a) composition analysis using procedures developed by Rostler and White (*129*), and (b) low-temperature ductility (39.2 F) at 1 and 5 cm/min. Correlations are reported between various ductility tests and penetration and absolute viscosity values. At the time of their report, and after approximately one year of service, no observable deterioration was reported.

SUMMARY

On the basis of information reported in the literature, it appears clear that some loss of serviceability in an asphaltic surfacing can be accounted for by long-term changes which occur in the asphalt, the mixture of asphalt and aggregate, and to a lesser extent the aggregate. Observations by Housel (*61*) and the Canadian Good Roads Association (*21*) show that overall pavement serviceability decreases continuously from the date of construction, independent of the adequacy of the structural design for the traffic. This loss is no doubt due in part to the less than perfect durability properties of the surfacing.

The general consensus of those organizations and individuals concerned with research relative to the durability of asphaltic surfacings indicates that a prime factor influencing this durability is the asphalt and its susceptibility to change with time. For the most part, research has indicated that the manifestation of deterioration is the accelerated cracking and raveling of the surfacing.

For many years, research efforts have concentrated on correlating so-called asphalt "hardening," or change in consistency, with the field performance of asphaltic surfacings. The greatest effort has been associated with measurements of penetration, ductility, viscosity, and softening point. Table 40 summarizes some of the critical values reported by various research investigations relative to these factors.

Although there is considerable argument as to the interpretation and real meaning of these various consistency measurements, and as to which test is best for predicting the critical condition of the asphalt binder, the message of these tests seems clear. Based on past experience with asphaltic surfacing pavements constructed in thicknesses of 2, 3, or 4 in., asphalts which "harden" rapidly tend to give service lives less than expected by the materials engineer. Hveem, Zube and Skog (*70*) have suggested new tests intended to provide controls on the rate at which asphalts harden. These tests emphasize measuring the consistency of asphalt after it has been subjected to accelerated laboratory weathering designed to simulate long-term aging within the asphaltic surfacing.

Research and field studies have consistently demonstrated the association between mix design, construction, and the durability of asphaltic mixes; specifically, film thickness, percent of voids, asphalt content, asphalt grade, aggregate surface characteristics, and permeability. To some extent, these factors are interrelated; however, they are not directly associated with the asphalt or aggregate properties, with the possible exception of film thickness. They are subject to some control by the engineer responsible for the design and construction of the asphaltic surfacing.

The factor of asphalt-aggregate adhesion, which plays a critical role in the performance of asphaltic surfacing, does not appear to be a serious problem, based on nationwide reports. However, Skog and Zube (*147*) do indicate that the problem may exist more often than suspected, at least in a localized area. Special tests have been suggested which should minimize the problem, providing such tests are used often enough to establish a pattern of performance.

Research consistently indicates the advantage of additives as a means of retaining the basic adhesive properties of a mix, particularly in the presence of water. In research papers it is emphasized that no single additive will work with all aggregates, and that laboratory testing is required to evaluate the potential benefits. One of the major difficulties in evaluating additives appears to be the lack of a good "stripping" test. Currently, it appears that an evaluation of strength or abrasion properties, after being subjected to the accelerated action of moisture or water, offers the best means of evaluation. Vallerga (*160*) discusses a surface additive or seal, which can be applied after mixes have been placed, that can act as an aid to slowing down the aging process. A variety of products of this type is available to the highway engineer; they offer a way of dealing with the durability problem for asphalts which are known to age rapidly.

Finally, some consideration must be given to the tentative implications expressed in Chapter Five, "Fatigue," such as: (a) for thin asphaltic surfacing, the softer asphalt provides the longer service life, and (b) for thick asphaltic surfacing, the harder asphalt provides the longer service life. If this proves to be true, it would appear that in the latter case asphalt hardening would not necessarily be undesirable except as it would affect raveling and resistance to water action.

DESIGN IMPLICATIONS

In spite of the tremendous research effort made to date, there does not appear to be a consensus as to how to eval-

TABLE 40

SUMMARY OF CRITICAL RECOVERED ASPHALT PROPERTIES FOR ACCEPTABLE PERFORMANCE OF ASPHALTIC SURFACING

REFERENCE	PENETRATION (0.1 MM)	DUCTILITY 77 F, 5 CM	DUCTILITY 45 F, 1 CM	SOFTENING POINT (°F)	VISCOSITY (POISES)
Dow	38 – 90				
Lewis and Welborn	< 20	< 10		> 150	
Hubbard and Gollomb	< 20				
Shattuck	< 20	< 25			
Thomas	< 20				
Hubbard	< 20				
Vokac	< 25	< 24		> 160	
Powers	< 10	< 10		> 160	
Halstead [a]	50	100			
	25	10			
Clark		< 15			
Doyle			< 8		
Hveem et al. [b]		16		151	
Simpson et al.					10^7–10^8 [b]

[a] Critical penetration (arithmetic)-ductility (logarithms) relationship as established by line connecting these values.
[b] Based on results of Zaca-Wigmore test project.

uate confidently the durability of an asphaltic surfacing prior to, or after, its placement. It appears that the great majority of such surfacings are durable and do perform in a satisfactory way. Some factors considered important to durability are as follows:

1. Mixes should be designed to provide for a maximum asphalt content without instability. This has long been axiomatic in mix design. The most positive way to attain this objective is to establish a total void requirement.

2. Mix designs should include minimum film thickness requirements. Campen et al. (*20*) have suggested a minimum film thickness of 6 microns (bitumen index of 1.23×10^{-3}). Based on the results of fracture strength research reported in Chapter Three, it appears that this value could be increased appreciably to about 20 microns, although further research is indicated. This could possibly be accomplished by adjusting aggregate grading requirements and using asphalts of higher mixing viscosity.

3. Mixes should be designed to have low permeability. Limiting criteria for air or water permeability are still being studied and require further evaluation. Goode and Lufsey (*41*) indicate this measurement may not be necessary, providing the voids are low. Until further evaluation is accomplished, it appears that use of the air permeability device can be a useful tool to adequate densification. Some useful information as to methods for measuring air permeability is given by Ellis and Schmidt (*33*) and Kari and Santucci (*75*).

4. Tests of physical properties, after exposure to water, should be performed on the asphaltic mixes in cases where performance history is unknown or suspect. The moisture vapor susceptibility or immersion-compression test (ASTM D1075) should be suitable until further research can develop a better test or tests.

5. Compaction of in-place asphaltic surfacing is critically important. In view of the evidence presented, a minimum compaction requirement should be specified. Many highway agencies now require a minimum relative density of 95 percent based on a specific laboratory compaction procedure. For airfield surfacing the minimum density is sometimes raised to 98 percent. Eventually, it would seem desirable to compact mixes initially to in-place voids contents of approximately 3 to 5 percent. Indications are that this condition would improve the long-range performance of asphaltic surfacing under almost every condition, provided the mix will remain stable.

6. The grade or consistency of the asphalt to be used appears to be a more controversial decision. It has been a general rule to use the asphalt of highest penetration (softest) possible, compatible with stability requirements. Several factors would tend to indicate this rule may require modification, at least under certain circumstances.

(a) Using a high penetration asphalt initially does not always assure a high penetration after mixing and 2 or 3 years of service. Some of the satisfactorily performing asphalts on the Zaca-Wigmore project had retained penetrations of only 25 percent of their original penetration after 30 and 35 months of service. Halstead indicates that even with high retained penetrations, if the ductility is low, the asphalt may not perform as expected.

(b) To obtain increased film thicknesses, the asphalt consistency may need to be relatively high.

(c) Based on limited fatigue data, asphalts of low penetrations or high viscosities may provide better fatigue properties when used in thick asphaltic surfacings (greater than 4 in.). This requires field verification; however, it appears to be worthy of consideration.

(d) Resistance to the effects of water may be increased by using asphalt of lower consistency. The available information does not extend to 40-50 or 60-70 penetration asphalts and should be researched further to include these grades.

In suggesting the harder asphalts, particularly for the thick asphaltic surfacing, the engineer must be mindful of the mixing and compaction requirements and, therefore, must balance the need to satisfy these requirements against stiffness, film thickness, etc., as previously discussed.

CHAPTER SEVEN

ASPHALTIC SURFACING REQUIREMENTS FOR STATIONARY LOADS, BRAKING AND ACCELERATION, AND ORTHOTROPIC BRIDGE DECKS

This chapter discusses briefly design factors for three loading conditions for which special designs are sometimes considered necessary. Specifically, these are:

1. Standing or stationary wheel loads.
2. Acceleration and deceleration loads.
3. Surfacings for orthotropic bridge decks.

STATIONARY WHEEL LOADS

Under stationary wheel loads it is generally considered that stability is the main mix requirement and that plastic (permanent) deformations are the undesirable manifestation of overstressing the mix, often as a result of standing wheel loads applied over long periods of time. It should be emphasized that special mix design factors, as described herein, are not necessarily required for asphaltic surfacings subjected to ordinary highway wheel loads. For these loadings, conventional Hveem or Marshall stability tests and mix design criteria should provide a mixture with sufficient resistance to deformation if properly designed and constructed. Other mix design procedures based on strength measurements and suitable field correlations would also be satisfactory.

For designing asphaltic concrete surfacings to be subjected to unusual environments or heavy stationary loads, special mix design procedures may be required. Two procedures have been suggested for use in such cases: (1) design procedures which measure the angle of friction, ϕ, and the cohesion, c, of the mix; and (2) design procedures which measure the stiffness modulus of the mix. Barber (*12*), Hewitt (*53*), McLeod (*90, 91*), Nijboer (*108*), and Smith (*149*) have presented analyses and recommendations in terms of the angle of friction and cohesion. Saal (*133*) has described the use of the stiffness modulus. Actually, these researchers were not concerned exclusively with developing mix design procedures for stationary loads. They were attempting to develop rational design methods which could be applied to any loading condition, of which stationary loads would be a special case.

McLeod (*90, 91*) developed a relationship defining the allowable vertical pressure in terms of angle of friction, cohesion, and lateral support provided by the pavement adjacent to the loaded area. The specific equation reported was

$$V = 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}\left[\frac{K(1+\sin\phi)+(1-\sin\phi)}{1-\sin\phi}\right] \qquad (42)$$

in which

$V =$ surface contact pressure;
$c =$ cohesion (or initial resistance) of bearing material;
$\phi =$ angle of internal friction of bearing material; and
$K =$ a constant related to lateral support.

Values of $K = 0$ and $K = ½$ are applicable for loads applied at or near the unsupported edge. $K = 1$ is applicable when no lateral support is assumed for the adjacent pavement. It seems reasonable, therefore, to use a K-value greater than 1 for most asphaltic concrete surfacings. However, until such time as more explicit data are available, McLeod has used the conservative value of $K = 1$. Figure 125 shows the combinations necessary for stability according to Eq. 42 when $K = 1$. McLeod (*90*) described a triaxial test which can be used for measuring ϕ and c for asphaltic mixtures. The rate of strain during loading in these tests ranges from 0 (equilibrium) to 0.4 in./min.

Hewitt (*53*) developed a relationship for design of flexible pavements based on the premise that the lateral stress produced in the pavement structure by surface loads must not exceed the passive lateral resistance provided by the components of the pavement structure, cohesion, c, and angle of internal friction, ϕ.

The equation as it applies to asphaltic surfaces is given as

$$p = \frac{4c}{1-\sin\phi}\left[\frac{1+\sin\phi}{1-\sin\phi}\right]^{1/2} \qquad (43)$$

in which

$p =$ surface contact pressure;
$\phi =$ angle of internal friction of bearing material; and
$c =$ cohesion of bearing material.

Values of ϕ were determined from Hveem stabilometer test data and values of c from the California cohesiometer test. Graphical representations of Eq. 43 relating values of ϕ and c required for stability as a function of traffic are shown in Figure 125. Because of the similarity in approach by McLeod and Hewitt, it is not surprising that the strength properties required for a loading condition of 100 psi (Eq. 43) are identical when a value of $K = 1$ is used in McLeod's relationship (Eq. 42).

Nijboer (*108*) has described the bearing capacity of sand-asphalt mixes by means of Prandtl's equation, as follows:

$$\sigma = \frac{\tau}{\tan\phi}\left[\frac{1+\sin\phi}{1-\sin\phi}e^{\tan\phi} - 1\right] \qquad (44)$$

in which

$\sigma =$ bearing capacity;
$\tau =$ initial resistance (cohesion); and
$\phi =$ angle of friction.

This equation is applicable for a plastic material of infinite thickness. For the case when $\phi = 0$, the bearing capacity can be estimated as follows:

$$\sigma = (2+\pi)\,\tau = 5.14\,\tau \qquad (45)$$

To determine ϕ and c, Nijboer recommends the triaxial compression test and a rate of strain equal to 0.005 in./min. Critical values of ϕ and c according to Nijboer are shown in Figure 125.

Smith (*149*) developed stability criteria based on the theory of elasticity for all types of bituminous mixes subjected to various traffic conditions. Utilizing the results of work by Barber and Mershon, Smith calculated the bearing strength of bituminous surfaces in terms of ϕ and c. On the basis of field observations, he determined criteria for satisfactory mixes as shown in Figure 125. The vertical boundary at $\phi = 25°$ was established on the basis that mixes exhibiting angles of internal friction less than 25° became overlubricated due to densification under traffic and, therefore, eventually may show signs of deformation. The lower boundary as identified by Smith is controlled by traffic conditions. As the traffic is decreased, the boundary between satisfactory and unsatisfactory mixes may be shifted downward as shown in Figure 125. Smith has recommended that a zero rate of strain (equilibrium) for triaxial tests be used to determine values for ϕ and c for the design of asphaltic surfacings.

All of the results presented so far are summarized in Figure 125. It can be noticed that the results of McLeod, Hewitt, and Smith, for a given loading condition, are in excellent agreement. Nijboer's analysis appears to allow much lower values for ϕ and c for similar loading conditions. The use of the Prandtl equation does not appear to account for this difference; therefore, for the present, no definite explanation is offered. One possibility is the difference between contact pressure as expressed by Nijboer. From the basic reference, this appears to be the inflation pressure increased by 50 percent. Thus, in terms of inflation pressure, the indicated 100-psi line would be decreased to 65 psi.

Saal (*133*) determined minimum requirements for the shear stress of asphaltic concrete and sand-asphalt at 50C (122 F) subjected to standing loads. The values of shear stress at failure reported by Saal were 3×10^5 dynes/cm^2 (4.35 psi) and 10^5 dynes/cm^2 (1.45 psi) for tire pressures of 9 kg/cm^2 and 4 kg/cm^2, respectively. In other words, the asphaltic mixture must exhibit a strength equal to these values in order to perform satisfactorily. Based on the shear strength requirement and an assumed strain of 1 percent, the minimum stiffness required to prevent excessive deformation can be estimated from

$$G = E/3 = \tau/\epsilon \qquad (46)$$

in which

G = shear modulus;
E = elastic modulus;
τ = shear stress; and
ϵ = shear strain.

For the condition of 9-kg/cm^2 (128-psi) tire pressure and 122 F, $E_{min} = (3 \times 4.35)/0.01 = 1{,}300$ psi; and for the condition of 4 kg/cm^2 and 122 F, $E_{min} = 435$ psi. Hence, for satisfactory performance, the stiffness of the pavement for long times of loading at 122 F should exceed these values.

Summary

For special loading situations in which stationary loads represent the prime design parameter and for which special design procedures are considered necessary, the method of triaxial testing seems particularly appropriate. Figure 125 suggests criteria which can be used to design asphaltic surfacings as a function of the angle of friction and cohesion for the selected requirement of contact pressure. It is emphasized that, in the opinion of the authors of this report, routine mix design procedures as represented by the

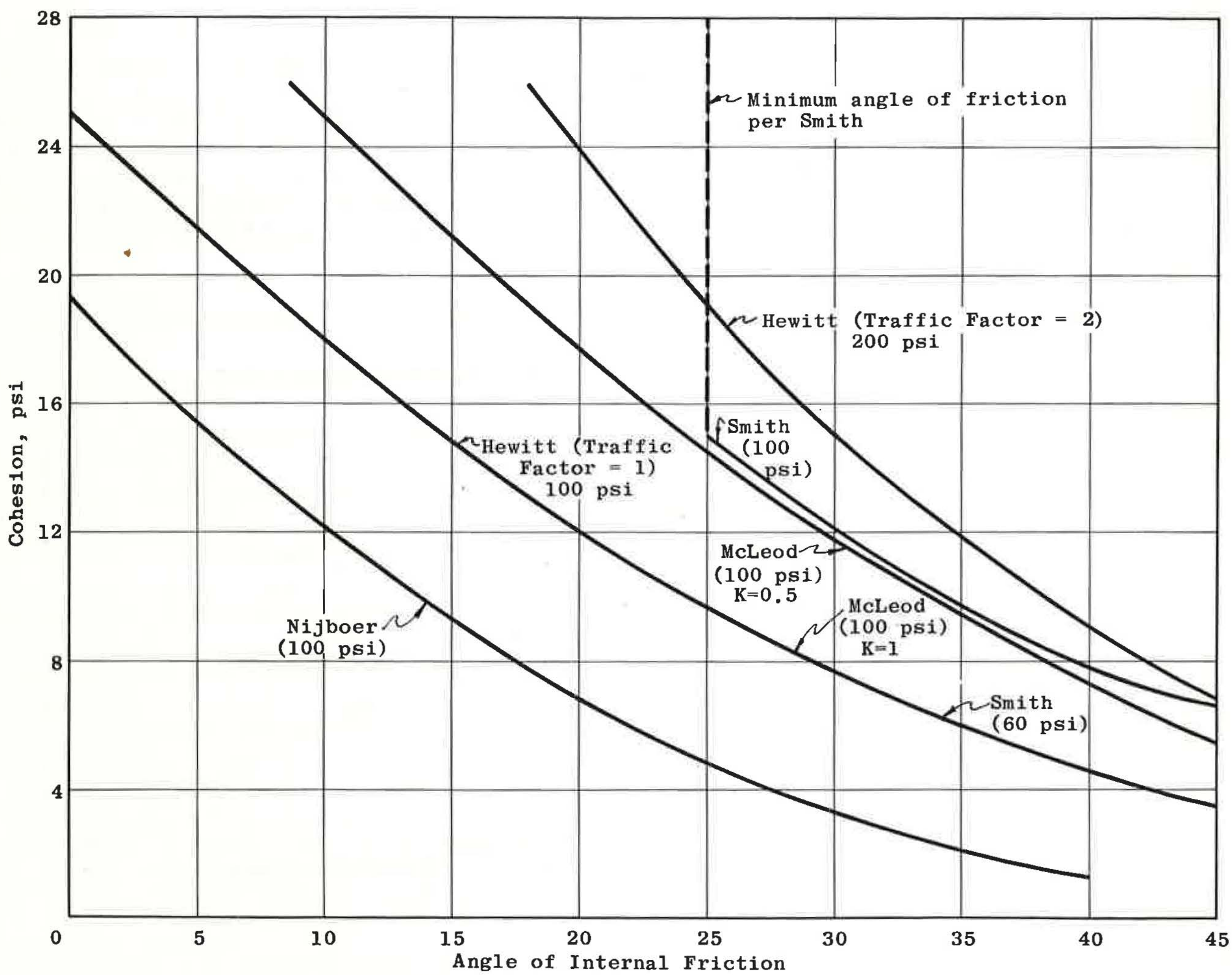


Figure 125. Asphaltic mix stability curves for static pressures.

Hveem and Marshall procedures, coupled with good construction, will provide a suitable surfacing under all but those cases in which exceptional loading conditions exist.

STABILITY UNDER BRAKING AND ACCELERATION STRESSES

Information relative to mix design requirements for acceleration and deceleration loading is extremely limited.

McLeod (*90*) has developed some relations which take into account the friction between the surfacing and the tire and between the surfacing and the base. The equations were derived on the assumption that to prevent pavement failure under a combination of vertical tire pressure and the braking stress, the sum of the applied stresses must not exceed the sum of the reactions that can be developed. Equations given by McLeod include the factors of V, the allowable contact pressure; c, the cohesion of the bearing material; ϕ, the angle of internal friction of the bearing material; K, the lateral support factor in the longitudinal direction; J, the lateral support factor in the transverse direction; l, the length of contact area in the longitudinal direction; f, the coefficient of friction between tire and surfacing; g, the coefficient of friction between surfacing and base; and $(p - q)$, the difference in horizontal stress at tire-surface interface and surface-base interface.

McLeod gives limiting design recommendations for contact pressures of 100 psi for a conservative (maximum) value of $(p - q) = 1.0$. Figure 126 illustrates the requirements for braking stresses and for slowly moving loads as suggested by McLeod's reports. It is evident that significantly higher strength properties are required for the braking and acceleration case as compared to the limiting curve for stationary or slow-moving loads as shown by the "limiting curve."

Barber (*12*) has analyzed the effect of horizontal stresses as regards the stability of layered systems. The effect of horizontal load components in reducing bearing capacity was evaluated, as was the combined effect of horizontal and vertical stresses.

Barber indicated that tangential forces up to about 10 percent of the vertical loads may be produced by longitudinal grades or superelevation, and that the ratio of tangential to normal loads is limited by the coefficient of friction between tire and surface, indicating that this ratio may reach a maximum value of about 0.8. Barber's analysis indicates that a substantial reduction in bearing capacity may occur in cases where horizontal stresses are involved. He indicated that a bituminous material having a bearing capacity of 555 psi under a vertical load is reduced to 132 psi at impending skidding. Barber has also presented information relative to the change in horizontal shear stress as a function of depth. On the basis of this information, it appears that the bond strength between two layers is critical to a depth of approximately one-half the equivalent contact radius of the loaded area. For most highway loading this would be about 4 in.

Summary

Information relative to special requirements for loading conditions associated with horizontal stresses appears to be limited. However, sufficient research has been completed to indicate that asphaltic surfacings subjected to horizontal loading should be designed for higher strength criteria. Although this may be an obvious qualitative conclusion, the usual difficulty is in quantifying the stress and strength

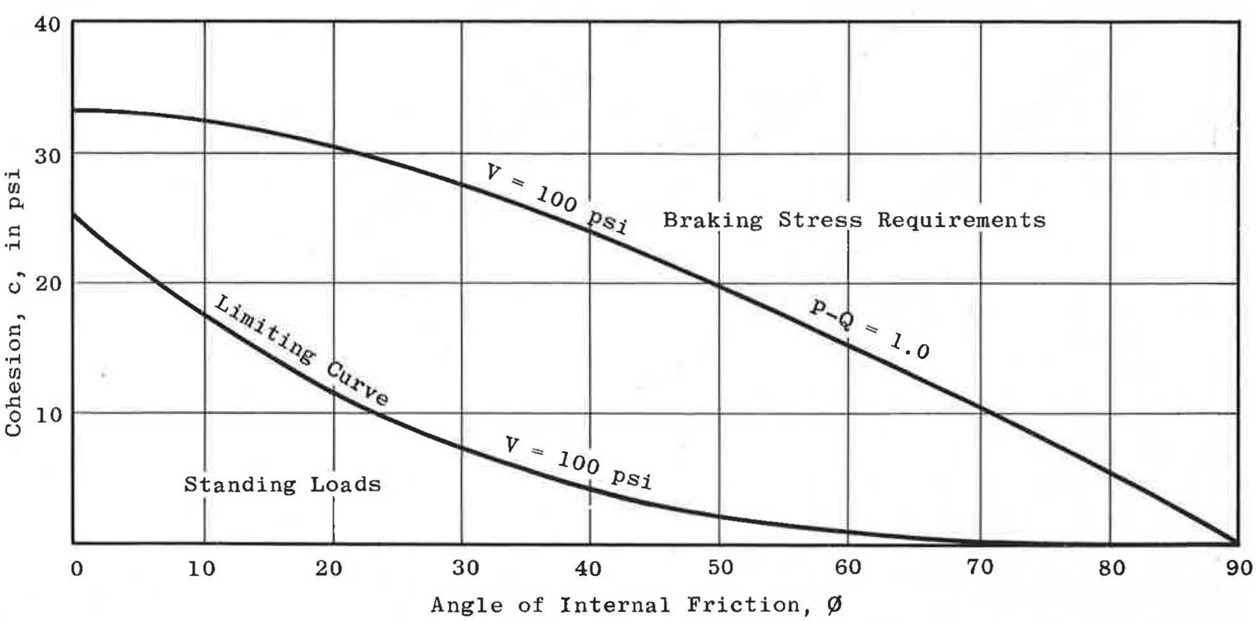


Figure 126. Illustrating stability curves for a given vertical load for bituminous paving mixtures subject to braking stresses (after McLeod (90) by permission).

parameters. To a limited extent, some guidance has been suggested by McLeod, Nijboer, and Barber. Use of this type of mix design criterion for a particular asphaltic surfacing will depend on the particular situation. It would be expected that such methods would be used only if existing designs are not performing as required or if unusual loadings are anticipated.

ASPHALTIC SURFACING FOR ORTHOTROPIC BRIDGE DECKS

The term "orthotropic bridge" as used in this report refers to bridges with steel roadways. The characteristic of orthotropic bridge roadways which makes them somewhat different from most steel roadways is the amount of deflection associated with the wheel loads. It has been suggested that this deflection could be as large as 3/32 in. over a 26-in. length. For this deflection, a 2½-in. layer of asphaltic concrete could develop a strain of approximately $3{,}000 \times 10^{-6}$ in. per inch and a 1-in. layer could develop a strain of approximately 800×10^{-6} in. per inch. In either case, the fatigue life expectancy would be relatively short. Hence, the problem of using asphaltic concrete for a surfacing will require special considerations.

The purpose of this section is primarily to point out some of the special problems more than to suggest materials to use. Some mention will be made as to materials and design which have been used or are planned for use.

Requirements for a steel bridge surfacing system have been described in the literature as follows:

1. Surfacing should be stable, unaffected by traffic and temperature, with good adhesion to the deck.
2. Surfacing should be lightweight, by use of either a thin section or lightweight material, or both.
3. Surfacing should be impervious to water and chemicals in order to prevent corrosion of the steel.
4. Surfacing must be flexible enough to resist cracking at relatively high levels of strain.
5. Surfacing must be skid resistant under expected service conditions.
6. Surfacing must be durable to resist changes in the desired properties with time.
7. Surfacing must be smooth riding, with a surface tolerance not to exceed ⅛ in. in 12 ft.
8. Surfacing must be easy to maintain and repair.

Three types of steel deck paving systems have been suggested in Europe (*37*)—the layered system, the stabilized Mastix system, and thin combination coatings. Only the first two systems have been used to any great extent.

The layered system consists of a prime coat to prevent corrosion, an insulation layer (Isolierung), a leveling course approximately 1 in. thick, and a surface course (Gussasphalt) also approximately 1 in. thick.

The prime coat is a thin organic or inorganic coating applied to the steel plate and serves as a bonding material for the subsequent layer, as well as to prevent corrosion.

The insulation layer is placed directly over the prime coat to provide an impermeable barrier, to absorb differential movements between the surface courses and the steel plate, and to serve as a heat shield during the period when the leveling course is applied. The insulation layer is generally either a thin sheet of metal foil cemented to the prime coat or ¼ in. of asphalt Mastix. Asphalt Mastix is a sand-asphalt mixture, 25-35 percent passing the No. 200 sieve, containing 14-25 percent of 15-60 penetration bitumen. Natural rubber is sometimes added to the bitumen.

The leveling course is either an asphalt binder or Gussasphalt. The asphalt binder is described as a coarse, open-graded mixture containing 4-6 percent of 60-70 penetration bitumen. Gussasphalt is a hard, dense, impermeable mix containing 7-10 percent of 20-40 penetration bitumen.

The surface course is usually Gussasphalt used to present a stable, skid resistant, and impermeable layer. To provide skid resistance, crushed stone (½ in.) is hand laid on the surface and rolled in before it hardens.

The stabilized Mastix consists of placing approximately 1½ in. of stabilized Mastix directly on a prime coat or thin insulating layer.

A four-component surfacing system selected for use on the Poplar Street Bridge in St. Louis, Mo., is composed of the following items:

1. A protective coat for the steel.
2. An air- and moisture-proof seal coat.
3. A 1½-in. asphaltic concrete leveling course.
4. A 1-in. asphaltic concrete wearing surface.

A test bridge was surfaced in 1962 to evaluate the proposed design. The steel deck plate of the test bridge was blast-cleaned to "white metal" and given a 5-mil metallizing coat of zinc. Two different tar epoxies were used for the seal coat, each on one-half of the bridge. Two spray coats were applied; in the second coat a sand grit was sprinkled in the wet epoxy to provide anchorage for the subsequent asphaltic courses. This was followed by a tack coat. The deck surface area was divided into four 6-ft wide strips extending parallel to traffic, and four types of asphaltic pavement were placed in approximately one-half length strips in each traffic lane. Two areas were chosen for the test sections, so that each type pavement was subjected to different plate deflection contours. The asphaltic pavement mixes were placed in two courses as mentioned previously. The mixes were: Type I, asphaltic concrete with liquid latex additives in both courses; Type II, asphaltic concrete with rubberized asphalt in both courses; Type III, leveling course same as Type II and wearing course of sand-asphalt with asbestos fiber and liquid latex additives; and Type IV, asphaltic concrete with asbestos fiber additive in both courses.

The report by Fondriest and Snyder (*37*) summarizes a study for the American Iron and Steel Institute relative to current paving practices for orthotropic bridge decks. The conclusions from this study relative to European practices were:

1. No ideal surfacing system has been found. However, pavings are possible that, with reasonable maintenance measures, can be expected to perform for ten years or more.
2. Corrosion of the steel deck plate has not been a problem. With reasonable care a number of materials will provide adequate protection against corrosion.
3. The use of a Mastix Isolierung has proved effective,

both for waterproofing and as a shear layer to prevent temperature cracks in the pavement.

4. A metal foil layer over the deck, although effective for waterproofing, may cause a number of other problems and is not recommended.

5. The two-layer wearing surface has been used most often, but tests show that a single layer can be as effective and has obvious economic advantages.

6. Tests have shown that an asphaltic pavement can contribute significantly to the stiffness of the steel plate, depending on the temperature and the speed of loading.

7. Very thin organic or inorganic single-coat paving systems have been little used to date, but show promise for the future.

Summary

It appears that several systems are potentially suitable for using asphaltic surfacings on orthotropic bridge decks. Experience on European constructions has provided a background of information from which to proceed. Experience and further research are required in order to refine the performance and material properties associated with this type of surfacing.

CHAPTER EIGHT

METHODS OF MEASUREMENT

In considering tests or test methods to be used to measure the characteristics of asphaltic mixtures, it is important to consider the following factors:

1. The properties or characteristics of the individual materials.
2. The planned use of the asphaltic mix.
3. The methods of analysis and application available.

In the foregoing items the key words are "characteristics," "use," and "analysis." The implication of these words is as follows:

1. Material "characteristics" refers to a prior knowledge of the general performance of the materials when used in asphaltic surfacing. For example, some geological formations are known to yield aggregates which are soft or are absorptive, and some asphalts are known to be somewhat "tender" immediately after construction. When information of this type is known, methods of measurement should be stipulated or developed to identify these properties. Material characteristics also refers to an awareness of the time-temperature tendencies of asphalt and asphaltic mixes.

2. Material "use" refers to such items as environment, structural design, types of loading, and tire pressure. A specific example would be the design of asphaltic surfacing for northern areas of the United States, where the average winter temperatures are extremely low and where thermal properties would be important.

3. The reference to methods of "analysis" is self-explanatory in that it refers to performing tests which will be meaningful in terms of available methods of interpretation.

One of the most important properties to be considered relative to testing asphaltic concrete is the time-temperature dependency of the asphalt and the asphaltic concrete. This factor has been discussed in each of the preceding chapters and will play a key role in subsequent discussions. In this regard it should be remembered that it will be a rare situation in which asphaltic surfacings are subjected to uniform annual temperatures or rates of loading. Therefore, it is necessary to consider in the test method a range of temperatures and rates of loading which will be representative of in-service conditions. These factors could vary, depending on the location and use of the pavement. In most cases, tests are performed at the most critical condition to be expected (e.g., stability tests are usually performed at 140 C, the maximum temperature to be expected in the upper levels of the surfacing). It should not be necessary to always select the most extreme single condition for testing—this will depend on the purpose of the test. For example, in fatigue testing some type of conservative seasonal average would seem to represent a more reasonable approach.

A factor which is very important to testing is the method used in preparing the test specimen. Good correlations between laboratory and field can only be expected if the specimen approximates the field condition. Recent efforts to develop the Triaxial Institute kneading compactor or the gyratory compactor represent a recognition of this need. Tests of asphalt durability in thin films is another example of recognizing the *in-situ* conditions. Although this item may not be stipulated in the general test methods discussed in this portion of the report, it is implied that test methods should always attempt to represent field conditions.

The remaining section of this chapter discusses methods of measurement of the various factors described in Chapters Two through Seven. In most cases no standard methods of measurement are available; therefore, it is not possible to be definitive. To be able to compare test results, future research should be directed to standardizing methods of measurement for such factors as stiffness, fracture strength, fatigue, durability, and shear strength.

STIFFNESS

A variety of procedures have been suggested in Chapter Two for measuring stiffness. Direct measurements include the following:

1. Creep.
2. Stress relaxation.
3. Constant rate of strain.
4. Complex modulus.
5. Flexure or bending.

An indirect method based on the properties of recovered asphalt and volume concentration of aggregate in asphaltic mixes was also suggested. The evidence presented in Chapter Two indicates that all of the direct methods are applicable as long as deformations remain small. Monismith et al. (*104*) have suggested a limiting strain of 0.1 percent to evaluate linear viscoelastic properties. The deviations which have been encountered between test methods by some investigators are considered well within present limitations for measuring stiffness, and certainly within present ability to apply theoretical analysis to field performance. Each direct method of testing has specific requirements which sometimes make the test difficult to perform without expensive and sophisticated equipment. Bending tests of the type used by Deacon (*25, 26*), Jimenez and Gallaway (*72*), Papazian and Baker (*112*), Pell (*120*), and others, have the unique advantage of simultaneously combining the tension and compression characteristics in a test which, to some degree, simulates field loading conditions. This type of test cannot be performed at high temperatures; however, the concept of time-temperature interchangeability can be used to extend the range of test results.

The creep test is considered to be the simplest method for routinely measuring the stiffness modulus of asphaltic concrete. This test is described by Monismith et al. (*104*) and by Pagen (*111*). The creep test can be performed in tension or compression. Basically, a specimen is subjected to a static stress in a controlled-temperature environment and the resulting strain-time curve is obtained. The stiffness is obtained from the ratio of the static stress to the strain at various times of loading. If high-frequency stiffness moduli are required, the test can be performed at subzero temperatures over convenient time intervals and then, by means of the time-temperature superposition concept, the stiffness at shorter time intervals at the higher temperatures can be reliably estimated. The minimum temperature to use would be that expected for the in-service surfacing. Stiffnesses have been measured up to 110 F by this method.

Regardless of the method of test, two factors should be considered in measuring stiffness if it is intended to use the theory of elasticity to compute stress and strain. First, the specimen should be conditioned by repetitive load applications at low levels of stress. For creep test, Pagen has reported that after 5 or 6 repetitions of load, a constant relationship develops between stress and strain. In bending tests, Monismith has suggested at least 100 repetitions. Second, computations of stiffness should be made within the range of linear viscoelastic response to load.

FRACTURE STRENGTH

The fracture strength characteristic of asphaltic concrete can be evaluated by test procedures described by Davis, Krokosky, and Tons (*24*), Monismith, Secor, and Secor (*103*), Hargett (*46*), and Hewitt (*53*). Tension testing of asphaltic concrete is limited by temperature; however, measurements have been made, on inclined planes, up to 100 F. The measurement of tensile strength should be related to the required performance. For example, if fracture under a single dynamic load is considered critical, tests should be made at loading times and temperatures representative of field conditions. If thermal stresses are considered critical, tests should be made at cold temperatures at low rates of strain.

THERMAL STRESS

A direct method for measuring thermal stress is described by Monismith, Secor, and Secor (*103*). Tests were made using 1-in. square bars 12 in. in length placed in a frame made of invar steel. The apparatus is comparatively rigid and thus is capable of restraining any deformation that may develop due to temperature change. By placing the frame in a controlled-temperature cabinet, it is possible to subject the specimen to a range of temperatures. Initially the specimen and frame are brought to equilibrium at an elevated temperature, after which the specimen is cemented to the frame and subjected to a change in temperature. Stresses are obtained by means of strain gauges attached to the invar frame.

Although alternate procedures are no doubt available, the method proposed by these investigators appears satisfactory for the purpose. It is emphasized that this is a measure of stress and not strength, as is generally associated with methods for measurement.

FATIGUE PROPERTIES

For purposes of fatigue, some type of bending test seems mandatory. No recommendation can yet be made relative to an exact procedure. Chapter Five describes several methods used by researchers in the United States and Europe. If comparisons of fatigue properties are of prime consideration, either constant stress or constant strain bending tests should be used. In this way, comparisons between laboratories can be made, making it possible to accumulate information from various sources. The equipment developed by Deacon (*25, 26*) is reasonably simple and can be operated in either mode of testing; therefore, it is recommended at this time. If quantitative comparisons with field performance are required, some procedure of introducing the interaction between the asphaltic surface and the underlying layers will be important. Jimenez and Gallaway (*72*) and Papazian and Baker (*112*) have suggested tests which may satisfy this requirement. It is important that fatigue tests be performed under dynamic conditions approaching slow-moving traffic. Load durations of 0.1 sec would seem to be a conservative approach to moving highway loads.

DURABILITY PROPERTIES

A discussion of test methods to be used for evaluating durability has been included in Chapter Six, and no specific recommendations are made. One of the prime weaknesses in most of this type of research effort is the lack of well-defined correlations with field performance. Krchma and Groening (*77*) cover this item with the following statement:

> ". . . first and foremost, laboratory tests and experiments, to be reliable and meaningful must be rigorously correlated with field performance. This has been missing and may be responsible for the present situation."

The Zaca-Wigmore Experimental Test Road is a notable exception to this statement and was an attempt to correlate laboratory tests with in-service pavement performance.

Based on field observations and correlations, ductility at in-service temperatures of the aged asphalt appears to be associated with the long-term performance of asphalt. Present test limits have been subject to considerable dispute. The correlation appears to be slightly improved by combining ductility with recovered penetration. Several researchers have indicated that chemical fractionation presents the best possibility for estimating asphalt durability. Some documentation for this argument has been presented by Rostler and White (*129*) and by Halstead, Rostler, and White (*45*).

The effect of water on asphaltic surfacing poses a durability problem for which there is no particular test. As a minimum requirement, most research engineers believe a measurement of strength properties after exposure to water is best. The ASTM immersion-compression test satisfies this requirement, as does the California MVS (moisture vapor susceptibility) test. Future consideration should be given to imposing this type of exposure on fatigue tests in order to obtain a better estimate of its effect on pavement damage due to repetitive loading.

Durability or long-term performance of an asphaltic surfacing has been shown to be significantly related to the in-place characteristics or condition of the asphaltic concrete. Therefore, the durability of an asphaltic surfacing made with an aggregate or asphalt of poor durability (changing properties with time) can be minimized by placing the asphaltic mix at a low void content with an adequate amount of asphalt and a low coefficient of air permeability. In this regard, the field measurement of air permeability may be a very useful tool for controlling construction. Similarly, air or water permeability may be useful in evaluating relative durability of in-service pavements. Each of these factors has been shown to be related to the durability of the surfacing.

STRENGTH PROPERTIES OF ASPHALTIC CONCRETE

The strength of asphaltic concrete in terms of its shearing resistance has been abundantly described in the literature (e.g., *4, 40, 94, 137, 149*). For many years some form of triaxial test has been used to measure the angle of friction and the cohesion of asphaltic concrete. The techniques discussed by Secor et al. (*137*) probably represent the most modern concepts relative to the method of interpretation. No specific recommendation for triaxial testing is given, as any of those described in the literature would no doubt be suitable for determining the angle of friction and a value of the cohesion. The key to the triaxial test will be the temperature and rate of loading used in the laboratory. The decision as to the temperature and rate of loading to be used should be guided by the field conditions.

SUMMARY

On the basis of the literature search, there appears to be no standardized methods or a composite method for evaluating the various factors discussed in this report. In each case, however, methods of testing are available which should provide useful data for the evaluation of the probable performance of the asphaltic surfacing. If the designer weighs the test results in the light of the use, past performance, and method of analysis, the reliability of satisfactory long-term performance should be adequate to meet most needs.

CHAPTER NINE

RECOMMENDATIONS FOR FUTURE RESEARCH

The purpose of this chapter is to recommend specific areas of future research. These recommendations are based on the review of literature and analyses made in connection with this report and are limited to the items discussed in Chapters Two through Seven. The emphasis has been placed on applied research rather than the development of basic or fundamental information. In this regard, it should not be implied that basic research is not considered necessary. This, of course, is not true, and basic research should continue wherever the talent and facilities are available. The mechanisms controlling stiffness and rheological properties of asphalt and asphaltic mixes require further study; the role chemical composition plays in durability and rheology should be explored, as should the mechanism of asphalt-aggregate adhesion. These are just a few of the areas where basic research is needed.

It would seem that the most fruitful applied research could be described as "verification" or "documentation" research. The implication of this terminology is the need for broad attempts to correlate laboratory research with well-documented field performance. This should include physical measurements (i.e., strain, temperature, etc.) within the pavement structure if such measurements can be helpful to the correlation.

It is emphasized that the research recommendations are directed to general uses of asphaltic surfacing rather than specialized uses, such as for orthotropic bridge decks or unique requirements associated with very heavy loads, very high tire pressures, or braking and acceleration. Also, research relative to mix design requirements has been excluded in view of the state of knowledge cited in Chapter One.

Research is recommended in the following areas:

1. Fatigue of asphaltic surfacings.
2. Durability of asphalt, aggregate, and asphaltic concrete.
3. Thermal stresses in asphaltic concrete.

The fatigue and durability research should take precedence over thermal stresses because of the broader application. However, in a fairly large section of the United States thermal stresses could be significantly influencing performance. Therefore, some research relating thermal stresses to performance is definitely needed.

FATIGUE OF ASPHALTIC SURFACING

Recommended areas of research concerning fatigue of asphaltic surfacing are as follows:

1. To establish the reliability of using laboratory fatigue measurements as a means of predicting the performance of asphaltic surfacings. This will require fatigue testing of both laboratory and field specimens, together with evaluation of traffic and performance. This investigation should assist in developing desirable and controllable properties for use in both structural design and mix design. A project of this type should also include a determination of the most appropriate type of fatigue test.

2. To investigate the possibility of using the stiffness modulus as a property of asphaltic concrete which can be reliably related to fatigue service life. Unfortunately, the information available is limited in scope. The main difficulty in using the stiffness modulus, *a priori,* as a measure of fatigue life is confounded by the lack of definitive information as to the effect of void content, aggregate gradation, asphalt content, asphalt grade, etc., for various types of mixes. For any specific asphaltic mix, the qualitative trend of stiffness to fatigue appears to be well established. The quantitative association needs more research. For example, two mixes of the same stiffness modulus measured on asphaltic specimens of widely different void content or gradations would probably not have the same fatigue service life.

3. To consider further the effect of complex field loading conditions on fatigue service life. Specifically, in-service asphaltic surfacings are subjected to the following loading conditions:

(a) Random loading (compound loading).
(b) Daily variations in temperature.
(c) Seasonal changes in temperature.
(d) Random variation in time of loading.
(e) Effect of aging with time.

It seems probable that the reliability of using fatigue data would be improved if these factors could be evaluated. The factor of compound loading has been investigated in a limited way by Deacon and apparently is not as significant as would be thought. It is suggested that each factor first be studied individually. Then, based on results, some attempts can possibly be made to evaluate the factors collectively. Again, *in-situ* measurements of stress, strain, and temperature should be included wherever possible.

4. To investigate the possibility of improved methods for measuring the fatigue properties of asphaltic surfacings. Two major factors require some further study. First, with the exception of Jimenez and Gallaway, present fatigue tests are based on uniaxial stresses and strains. The Poisson effects are minimized in this type of testing. Fatigue tests with biaxial stress and strain should be studied to determine if important influences are being overlooked. Second, in order to improve the quantitative correlations with in-service fatigue life of asphaltic surfacings, it may be necessary to introduce the effects of the interaction of the underlying material. The procedures of Papazian and Baker or Jimenez and Gallaway tend to do this; however, the interpretation in terms of limiting stress and strain values is somewhat confounded in this type of test.

5. To develop recommended procedures which will tend to optimize the fatigue service life of asphaltic surfacings. This investigation would essentially be the fruition of the research efforts described in items 1 through 4. These procedures should include both mix design and construction requirements.

DURABILITY OF ASPHALT, AGGREGATE, AND ASPHALTIC CONCRETE

Factors recommended for study concerning the durability of asphalt, aggregate, and asphaltic concrete are as follows:

1. To develop reliable determinants (test parameters) for estimating durability (long-term performance) of asphaltic surfacing. A tremendous amount of effort has been expended on this type of research in the past. Most of this effort has been concentrated in the laboratory, with some field performance observations during construction and the first few years of service life. In order to develop determinants for asphaltic surfacing durability, techniques must be investigated which will evaluate the performance of these surfacings as separated from the remaining portions of the structure. Two areas of major importance requiring immediate attention are as follows:

(a) The influence of asphalt properties on durability, particularly the influence of consistency or stiffness properties as a function of the thickness and structural performance of the surfacing.
(b) The influence of interactions of asphalt and aggre-

gate on durability, particularly in terms of the effects of water.

2. Based on the results of item 1, suitable tests should be developed which will identify the durability determinants. It is recommended that some type of laboratory performance test be used. An example might be to perform strength tests or fatigue tests with artificially aged materials. These tests are difficult, expensive, and time consuming, and should be used only as a basis for developing less complicated tests (such as the shot abrasion or pellet abrasion test). The tests mentioned are merely examples, not recommendations.

3. To study the influence of construction on durability. Requirements for void content, permeability (air or water), and asphalt film thickness have been associated with durability. Definitive criteria for use in standard specifications are needed.

SUMMARY

In summary, there is a need to improve or verify the correlation between field performance and laboratory tests, particularly in the areas of fatigue service life and durability. Until these correlations are performed, the benefits from continued laboratory studies will be questionable. Similar studies of the effect of thermal stress on performance as predicted from laboratory tests and analytical analysis are also required.

An experiment which might be used to develop some performance information relative to fatigue service life is described in the Appendix. Similar type projects might also be developed for durability and thermal stress investigations. It is not suggested that this represents the only way of satisfying the requirements for studies of fatigue. It is, however, a way which the authors consider would be productive of useful information.

REFERENCES

1. Abbott, F. P., and Craig, W. G., "Determination of Age Hardening Tendencies and Water Susceptibility of Paving Asphalt by the Sonic Method." *HRB Bull. 270* (1960) pp. 20-37.
2. Abraham, H., *Asphalts and Allied Substances,* Vol. 2. Van Nostrand (1945).
3. Alexander, R. L., "Limits of Linear Behavior of an Asphalt Concrete in Tension and Compression." Thesis, D. Eng. (Transportation Eng.), Univ. of California (1964).
4. *Triaxial Testing of Soils and Bituminous Mixtures. Spec. Tech. Publ. No. 106,* ASTM (1951), 303 pp.
5. *Symposium on Effect of Water on Bituminous Paving Mixtures. Spec. Tech. Publ. No. 240,* ASTM (1958).
6. *Symposium on Road and Paving Materials—1959. Spec. Tech. Publ. No. 277,* ASTM (1959), 126 pp.
7. "Digging Deep for Temperature Information." *Asphalt,* Asphalt Inst. (Oct. 1964).
8. "Symposium on Hardening of Paving Asphalts." *Proc. Assn. of Asphalt Paving Tech.,* Vol. 27 (1958) pp. 171-231.
9. "Symposium on Flexible Pavement Behavior as Related to Deflection." *Proc. Assn. of Asphalt Paving Tech.,* Vol. 31 (1962) pp. 208-399.
10. "Index." *Proc. Assn. of Asphalt Paving Tech.,* Vol. 31 (1962) pp. 627-686.
11. Barber, E. S., "Calculation of Maximum Pavement Temperatures from Weather Reports." *HRB Bull. 168* (1957) pp. 1-8.
12. Barber, E. S., "Shear Loads on Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 354-357.
13. Benkelman, A. C., "Analysis of Flexible Pavement Deflection and Behavior Data." *HRB Bull. 210* (1958) pp. 39-48.
14. Benkelman, A. C., and Williams, S., "The Structural Behavior of Flexible Pavement—An Analysis of Rigid-Plate Bearing Tests on Full-Size Test Sections." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 686-712.
15. Benkelman, A. C., Kingham, R. I., and Fang, H. Y., "Special Deflection Studies on Flexible Pavement." *HRB Spec. Rep. 73* (1962) pp. 102-120.
16. Brodnyan, J. G., "Use of Rheological and Other Data in Asphalt Engineering Problems." *HRB Bull. 192* (1958) pp. 1-19.
17. *"Thermal Properties of Concrete (Bull. 1), Part VII."* Project Final Report, U. S. Bur. of Reclamation (1940).
18. *California Division of Highways Planning Manual.* Part 7, "Design" (1963).
19. "Stresses and Deflections in Foundations and Pavements." Univ. of California, Dept. of Civil Eng. and Inst. of Transportation and Traffic Eng.
20. Campen, W. H., Smith, J. R., Erickson, L. G., and Mertz, L. R., "The Relationship Between Voids, Surface Area, Film Thickness and Stability in Bituminous Paving Mixtures." *Proc. Assn. Asphalt Paving Tech.,* Vol. 28 (1959) pp. 149-178.
21. Canadian Good Roads Assn., "Pavement Evaluation Studies in Canada." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 137-218.

22. Coffman, B. S., Kraft, D. C., and Tamayo, J., "A Comparison of Calculated and Measured Deflections for the AASHO Test Road." *Proc. Assn. Asphalt Paving Tech.*, Vol. 33 (1964) pp. 54-91.
23. Critz, P. F., "Laboratory Study of Anti-Stripping Additives for Bituminous Materials." *Spec. Tech. Publ. No. 240,* ASTM (1958) pp. 48 ff.
24. Davis, E. F., Krokosky, E. M., and Tons, E., "Stress Relaxation of Bituminous Concrete in Tension." *Hwy. Res. Record No. 67* (1965) pp. 38-58.
25. Deacon, J. A., "Fatigue of Asphalt Concrete." Thesis, D. Eng. (Transportation Eng.), Univ. of California (1964).
26. Deacon, J. A., and Monismith, C. L., "Laboratory Flexural-Fatigue Testing of Asphalt-Concrete with Emphasis on Compound-Loading Tests." *Hwy. Res. Record No. 158* (1967) pp. 1-31.
27. Dehlen, G. L., "Flexure of a Road Surfacing, Its Relation to Fatigue Cracking, and Factors Determining Its Severity." *HRB Bull. 321* (1962) pp. 26-39.
28. Dormon, G. M., "The Extension to Practice of a Fundamental Procedure for the Design of Flexible Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 785-793.
29. Dormon, G. M., and Metcalf, C. T., "Design Curves for Flexible Pavements Based on Layered System Theory." *Hwy. Res. Record No. 71* (1965) pp. 69-84.
30. Doyle, P. C., "Cracking Characteristic of Asphalt Cement," *Proc. Assn. of Asphalt Paving Tech.,* Vol. 27 (1958) pp. 581-597.
31. Doyle, P. C., "Yardstick for Guidance in Evaluating Quality of Asphalt Cement." *Hwy. Res. Record No. 24* (1963) pp. 164-181.
32. Dunlap, W. A., and Stark, L. E., "Deflection Tests on Texas Highways." *HRB Bull. 269* (1960) pp. 52-65.
33. Ellis, W. H., and Schmidt, R. J., "A Method for Measuring Air Permeabilities of Asphalt Concrete Pavements." *Spec. Tech. Publ. No. 294,* ASTM (1960).
34. Eriksson, R., *Deformation and Strength of Asphalts at Slow and Rapid Loadings.* Statens Vaginstitut, Moddelance 82, Stockholm (1951).
35. Eriksson, R., *Tension Tests on Sheet-Asphalt.* Statens Vaginstitut, Moddelance 82, Stockholm (1954).
36. Fisher, J. M., Lee, A. R., and Millard, R. S., "Research and Its Application to Road Construction." *Proc. Inst. Civil Eng.,* Vol. 12 (1959).
37. Fondriest, F. F., and Snyder, M. J., *Current Paving Practices on Orthotropic Bridge Decks.* Battelle Memorial Inst., Bull. No. 1 (Oct. 1965).
38. Gallaway, B. M., "Factors Relating Chemical Composition and Rheological Properties of Paving Asphalts with Durability." *Proc. Assn. Asphalt Paving Tech.,* Vol. 28 (1959) pp. 280-294.
39. Garrison, W. A., "Three-Year Evaluation of Shell Avenue Test Road." *Hwy. Res. Record No. 117* (1966) pp. 71-99.
40. Goetz, W. H., and Chen, C., "Vacuum Triaxial Technique Applied to Bituminous-Aggregate Mixtures." *Proc. Assn. Asphalt Paving Tech.,* Vol. 19 (1950) pp. 55-81.
41. Goode, J. F., and Lufsey, L. A., "Voids, Permeability, Film Thickness vs. Asphalt Hardening." *Proc. Assn. Asphalt Paving Tech.,* Vol. 34 (1965) pp. 430-463.
42. Gotolski, W. H., Ciesielski, S. K., and Heagy, L. N., "Progress Report on Changing Asphalt Properties of In-Service Pavements in Pennsylvania." *Proc. Assn. Asphalt Paving Tech.,* Vol. 33 (1964) pp. 285-319.
43. Griffin, R. L., Simpson, W. C., and Miles, T. K., "Influence of Composition of Paving Asphalt on Viscosity, Viscosity-Temperature Susceptibility and Durability." *Symposium on Chemistry and Composition of Asphalts,* ACS (1958).
44. Halstead, W. J., "The Relation of Asphalt Ductility to Pavement Performance." *Proc. Assn. Asphalt Paving Tech.,* Vol. 32 (1963) pp. 247-271.
45. Halstead, W., Rostler, F. S., and White, R. M., "Properties of Highway Asphalts—Part III, Influence of Chemical Composition." *Proc. Assn. Asphalt Paving Tech.,* Vol. 35 (1966) pp. 91-138.
46. Hargett, E. R., "Basic Material Properties for the Design of Bituminous Concrete Surfaces." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 606-610.
47. Hein, T. C., and Schmidt, R. J., "Air Permeability of Asphalt Concrete." *Spec. Tech. Publ. No. 309,* ASTM (1961) pp. 49-62.
48. Heithaus, J. J., and Johnson, R. W., "A Microviscometer Study of Road Asphalt Hardening in the Field and Laboratory." *Proc. Assn. Asphalt Paving Tech.* (1958) pp. 17-34.
49. Heukelom, W., "Analysis of Dynamic Deflections of Soils and Pavements." *Geotech.,* Vol. 11 (Sept. 1961).
50. Heukelom, W., and Klomp, A. J. G., "Dynamic Testing as a Means of Controlling Pavements During and After Construction." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 667-679.
51. Heukelom, W., and Klomp, A. J. G., "Road Design and Dynamic Loading." *Proc. Assn. Asphalt Paving Tech.,* Vol. 33 (1964) pp. 92-125.
52. Hewitt, W. L., "Analysis of Stresses in Flexible Pavements and Development of a Structural Design Procedure." *HRB Bull. 269* (1960) pp. 66-74.
53. Hewitt, W. L., "Analysis of Flexible Paving Mixtures by Theoretical Design Procedure Based on Shear Strength." Hwy. Res. Record No. 104 (1965) pp. 78-104.
54. Hicks, L. D., "Flexible Pavement Deflection Study in North Carolina." *HRB Proc.,* Vol. 39 (1960) pp. 403-415.
55. Hicks, R. G., "Fatigue of Asphalt Concrete." Thesis, M. Eng., Univ. of California (1965).
56. "Supplemental Bibliography on Characteristics of

Bituminous Materials." Supplement to *HRB Bibl. 35* (Nov. 1964).
57. "The ASSHO Road Test, Report 5, Pavement Research." *HRB Spec. Rep. 61E* (1962).
58. "The WASHO Road Test, Part 2, Test Data, Analyses, Findings." *HRB Spec. Rep. 22* (1955).
59. Hogbin, L. B., "The Heat Capacity of Some Road Materials." *Res. Note No. RN/4082/LBR*, Brit. Road Res. Lab. (Nov. 1961).
60. Hooks, C. C., and Goetz, W. H., "Laboratory Thermal Expansion Measuring Techniques Applied to Bituminous Concrete." Report to U. S. Army Engineers, Waterways Experiment Station, Vicksburg, Miss., by Purdue Univ. (Aug. 1964).
61. Housel, W. S., "The Michigan Pavement Performance Study for Design Control and Serviceability Rating." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 219-253.
62. Housel, W. S., "Pavement Deflection as Related to the Ultimate Capacity of Flexible Pavements." *Proc. Assn. Asphalt Paving Tech.*, Vol. 31 (1962) pp. 377-399.
63. Hubbard, P., and Gollomb, H., "The Hardening of Asphalt with Relation to Development of Cracks in Asphalt Pavements." *Proc. Assn. Asphalt Paving Tech.*, Vol. 9 (Dec. 1937) pp. 165-194.
64. Hveem, F. N., "Quality Tests for Asphalts—A Progress Report." *Proc. Assn. Asphalt Paving Tech.*, Vol. 15 (1943) pp. 111-152.
65. Hveem, F. N., and Carmany, R. M., "The Factors Underlying the Rational Design of Pavements." *HRB Proc.*, Vol. 28 (1948) pp. 101-136.
66. Hveem, F. N., "Pavement Deflections and Fatigue Failures." *HRB Bull. 114* (1955) pp. 43-87.
67. Hveem, F. N., Zube, E., and Skog, J., "Progress Report on the Zaca-Wigmore Experimental Asphalt Test Project." *Spec. Tech. Publ. No. 277*, ASTM (1959).
68. Hveem, F. N., "Closing Statement by Chairman of Session VIII." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 1019-1022.
69. Hveem, F. N., Zube, E., and Bridges, R., "The Effects of Resilience-Deflection Relationship on the Structural Design of Asphalt Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 649-666.
70. Hveem, F. N., Zube, E., and Skog, J., "Proposed New Tests and Specifications for Paving Grade Asphalts." *Proc. Assn. Asphalt Paving Tech.*, Vol. 32 (1963) pp. 271-327.
71. James, J. G., "Epoxy Resins as Binders for Road and Bridge Surfacings." *Roads and Road Constr.* (Aug. 1963).
72. Jimenez, R. A., and Gallaway, B. M., "An Investigation of the Effects of a Pine Tar Extract on the Physical and Chemical Properties of Paving Grade Asphalt." *Proc. Assn. Asphalt Paving Tech.*, Vol. 29 (1960) pp. 290-315.
73. Jimenez, R. and Gallaway, B., "Behavior of Asphaltic Concrete Diaphragms to Repetitive Loadings." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 339-344.
74. Jimenez, R., and Gallaway, B., "Preliminary Report of an Apparatus for the Testing of Asphaltic Concrete Diaphrams." *Proc. Assn. Asphalt Paving Tech.* (1962) pp. 477-506.
75. Kari, W. J., and Santucci, L. E., "Control of Asphalt Concrete Construction by the Air Permeability Test." *Proc. Assn. Asphalt Paving Tech.*, Vol. 32 (1963) pp. 148-170.
76. Kenis, W. J., Sr., "Progress Report on Changes in Asphaltic Concrete in Service." *HRB Bull. 333* (1962) pp. 39-65.
77. Krchma, L. C., and Groening, T., "Influence of Pavement Voids, Asphalt Content and Asphalt Grade on Asphalt Performance." *Proc. Assn. Asphalt Paving Tech.*, Vol. 28 (1959) pp. 34-52.
78. Krokosky, E. M., and Chen, J. P., "Viscoelastic Analysis of the Marshall Test." *Proc. Assn. Asphalt Paving Tech.*, Vol. 33 (1964) pp. 406-436.
79. Krokosky, E. M., Tons, E., and Andrews, R. D., "Rheological Properties of Asphalt-Aggregate Compositions." *Proc. ASTM*, Vol. 63 (1963) pp. 1263-1286.
80. Lee, A. R., and Markwick, A. H. D., "The Mechanical Properties of Bituminous Surfacing Materials Under Constant Stress." *Jour. Soc. Chem. Ind.*, London (1937).
81. Lethersich, W., "Determination of Plastic and Brittle State of 'Pool' Bitumens." *Jour. Soc. Chem. Ind.*, Vol. 65, No. 7, pp. 190-196 (July 1946).
82. Lewis, R. H., and Welborn, J. Y., "A Study of the Effect of Characteristics of Asphalts on the Physical Properties of Bituminous Mixtures." *Proc. Assn. Asphalt Paving Tech.*, Vol. 17 (1948) pp. 228-263.
83. Lewis, R. H., and Welborn, J. Y., "The Properties of the Residues of 50-60 and 85-100 Penetration Asphalts from Oven Tests and Exposure." *Pub. Roads* (Apr. 1941) pp. 27-48.
84. Majidzadeh, K., and Herrin, M., "Modes of Failure and Strength of Asphalt Films Subjected to Tensile Stress." *Hwy. Res. Record No. 67* (1965) pp. 98-121.
85. Mack, C., "Physical Properties of Asphalts in Thin Films." *Ind. Eng. Chem.*, Vol. 49, No. 3, pp. 422-427 (Mar. 1957).
86. Mack, C., "Physical Chemistry." In *Bituminous Materials: Asphalts, Tars, and Pitches*," Vol. I, A. J. Hoiberg, ed. Interscience (1964).
87. Mack, C., "An Appraisal of Failure in Bituminous Pavements." *Proc. Assn. Asphalt Paving Tech.*, Vol. 34 (1965) pp. 234-247.
88. Marsh, A. D., "Measurements of the Specific Heats of Soils." *Res. Note No. RN/2189/ADM*, Brit. Road Res. Lab. (May 1954).
89. Marks, L. S., *Mechanical Engineer's Handbook.* McGraw-Hill (1951).
90. McLeod, N. W., "A Rational Method of Design for

Bituminous Paving Mixtures." *Proc. Assn. Asphalt Paving Tech.*, Vol. 19 (1950) pp. 82-224.

91. McLeod, N. W., "Rational Design of Bituminous Paving Mixtures with Curved Mohr Envelopes." *Proc. Assn. Asphalt Paving Tech.*, Vol. 21 (1952) pp. 349-437.
92. Mitchell, L. J., "Thermal Expansion Tests on Aggregates, Neat Cements, and Concretes." *Proc. ASTM*, Vol. 53 (1953) pp. 963-977.
93. Moavenzadeh, F., and Goetz, W. H., "Aggregate Degredation in Bituminous Mixtures." Hwy. Res. Record No. 24 (1963) pp. 106-137.
94. Monismith, C. L., and Vallerga, B. A., "Relationship Between Density and Stability of Asphaltic Paving Mixtures." *Proc. Assn. Asphalt Paving Tech.*, Vol. 25 (1956) pp. 88-108.
95. Monismith, C. L., "Flexibility Characteristics of Asphaltic Paving Mixtures." *Proc. Assn. Asphalt Paving Tech.*, Vol. 27 (1958) pp. 74-106.
96. Monismith, C. L., "Effect of Temperature on the Flexibility Characteristic of Asphaltic Paving." *Spec. Tech. Publ. No. 277*, ASTM (1959).
97. Monismith, C. L., Secor, K. E., and Blackmer, E. W., "Asphalt Mixture Behavior in Repeated Flexure." *Proc. Assn. Asphalt Paving Tech.*, Vol. 30 (1961) pp. 188-222.

97a. Monismith, C. L., *Asphalt Paving Mixtures—Properties, Design and Performance*. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley (1961).

98. Monismith, C. L., and Secor, K. E., "Viscoelastic Behavior of Asphalt Concrete Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 476-498.
99. Monismith, C. L., "Asphalt Mixture Behavior in Repeated Flexure." *IER Rep. No. TE-63-2* (to California Div. of Highways), Univ. of California (1963).
100. Monismith, C. L., "Asphalt Mixture Behavior in Repeated Flexure." *IER Rep. No. TE-64-2* (to California Div. of Highways), Univ. of California (1964).
101. Monismith, C. L., "Asphalt Mixture Behavior in Repeated Flexure." *IER Rep. No. TE-65-9*, Univ. of California (Nov. 1965).
102. Monismith, C. L., Secor, G.A., Secor, K. E., "Temperature Induced Stresses and Deformations in Asphalt Concrete." *Proc. Assn. Asphalt Paving Tech.*, Vol. 34 (1965) pp. 248-285.
103. Monismith, C. L., "Fatigue of Asphalt Paving Mixtures." Paper presented at First Annual Street and Highway Conf., Univ. of Nevada (1966).
104. Monismith, C. L., Alexander, R. L., and Secor, K. E., "Rheologic Behavior of Asphalt Concrete." *Proc. Assn. Asphalt Paving Tech.*, Vol. 35 (1966) pp. 400-450.
105. Morgan, A. D., "Service Record Study of Bituminous Concrete and Sand Asphalt Pavements in North Carolina." *HRB Bull. 154* (1957) pp. 21-30.
106. Nichols, F. P. Jr., "Effects of Compaction and Subgrade Stabilization on Deflections and Performance of Virginia Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 581-591.
107. Nijboer, L. W., "Mechanical Properties of Asphalt Materials and Structural Design of Asphalt Roads." *HRB Proc.*, Vol. 33 (1954) pp. 185-200.
108. Nijboer, L. W., *Plasticity as a Factor in the Design of Bituminous Road Carpets*. Elsevier (1948).
109. Pagen, C. A., and Ku, B., "Effect of Asphalt Viscosity on Rheological Properties of Bituminous Concrete." *Hwy. Res. Record No. 104* (1965) pp. 124-140.
110. Pagen, C. A., "A Study of the Temperature-Dependent Rheological Characteristics of .Asphaltic Concrete." *Hwy. Res. Record No. 158* (1967) pp. 116-143.
111. Pagen, C. A., "Rheological Response of Bituminous Concrete." *Hwy. Res. Record No. 67* (1965) pp. 1-26.
112. Papazian, H. S., and Baker, R. F., "Analyses of Fatigue Type Properties of Bituminous Concrete." *Proc. Assn. Asphalt Paving Tech.*, Vol. 28 (1959) pp. 179-210.
113. Papazian, H. S., "The Response of Linear Viscoelastic Materials in the Frequency Domain with Emphasis on Asphaltic Concrete." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 454-463.
114. Parr, W., and Serafin, P., "Four Years Test Trends on the Michigan Bituminous Experimental Road." *Proc. Assn. Asphalt Paving Tech.*, Vol. 28 (1959) pp. 295-315.
115. Peattie, K. R., "A Fundamental Approach to the Design of Flexible Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 403-411.
116. Peattie, K. R., "Stress and Strain Factors for Three-Layer Elastic Systems." *HRB Bull. 342* (1962) pp. 215-253.
117. Perry, J. H., and Perry, R. H., *Engineering Manual*. McGraw-Hill (1959).
118. Pell, P. S., "Fatigue Characteristics of Bitumen and Bituminous Mixes." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 310-323.
119. Pell, P. S., McCarthy, P. F., and Gardner, R. R., "Fatigue of Bitumen and Bituminous Mixes." *Internat. Jour. Mech. Sci.* (1961).
120. Pell, P. S., "Fatigue of Bituminous Materials in Flexible Pavements." *Proc. Inst. Civil Eng.*, Vol. 31 (July 1965).
121. Pell, P. S., and McCarthy, P. F., "Amplitude Effect on Stiffness of Bitumen and Bituminous Mixes Under Dynamic Conditions." *Rheologia Acta*, Vol. 2, No. 2 (1962).
122. Pfeiffer, J. P., *The Properties of Asphaltic Bitumen*. Elsevier (1956).
123. Porter, O. J., "Development of CBR Flexible Pavement Design Method for Airfields: A Symposium." *Trans. ASCE*, Vol. 115 (1950) pp. 461-467.

124. Reichert, J., "Influence of Repeated Loading on the Deflection of Road Layer." *Proc. Fifth Internat. Conf. on Soils Mechanics and Foundation Eng.*, Vol. 2 (1961).

125. Rigden, P. J., and Lee, A. R., "The Brittle Fracture of Tars and Bitumens." *Jour. Appl. Chem.*, Vol. 3 (Feb. 1953).

126. *Bituminous Materials in Road Construction.* Brit. Road Res. Lab., H.M. Stat. Off., London (1962).

127. Rogers, C. F., Cashell, H. D., and Irick, P. E., "Nationwide Survey of Pavement Terminal Serviceability." *Hwy. Res. Record No. 42* (1963) pp. 26-40.

128. Rostler, F. S., and White, R. M., "Influence of Chemical Composition of Asphalts on Performance, Particularly Durability." *Spec. Tech. Publ. No. 277,* ASTM (1959) pp. 64-88.

129. Rostler, F. S., and White, R. M., "Composition and Changes in Composition of Highway Asphalts, 85-100 Penetration Grade." *Proc. Assn. Asphalt Paving Tech.*, Vol. 31 (1962) pp. 35-89.

130. Rostler, F. S., "Prediction of Performance Based on Asphalt Composition Using Precipitation Methods." *Proc. Highway Conf. on Research and Development,* Bur. of Public Roads (1965).

131. Saal, R. N. J., "III. Physical Properties of Asphaltic Bitumen; Surface Phenomena, Thermal and Electrical Properties, Etc." In *The Properties of Asphaltic Bitumen,* Elsevier (1950).

132. Saal, R. N. J., and Pell, P. S., "Fatigue Characteristics of Bituminous Road Mixes." Brit. Soc. of Rheology (1959).

133. Saal, R. N. J., "Mechanics of Technical Applications of Asphalt." Paper presented at Div. of Petroleum Chemistry, Am. Chem. Soc. (New York) (1960).

134. Schmidt, R. J., Boynton, B. F., and Santucci, L. B., "Dilatometric Determination of the Glass Transition Temperature of Asphalts and Amorphous Polymers." Paper presented at fall meeting, Petroleum Div., Am. Chem. Soc., Atlantic City, N. J. (Sept. 1965).

135. Schweyer, H. E., Chelton, H., and Brenner, H. H., "A Chromatography Study of Asphalt." *Proc. Assn. Asphalt Paving Tech.*, Vol. 24 (1955) pp. 3-30.

136. Schweyer, H. E., "Asphalt Composition and Properties." *HRB Bull. 192* (1958) pp. 33-62.

137. Secor, K. E., and Monismith, C. L., "Analysis of Triaxial Test Data on Asphalt Concrete Using Viscoelastic Principles," *Proc. HRB,* Vol. 40 (1961) pp. 295-314.

138. Secor, K. E., and Monismith, C. L., "Analysis and Interrelation of Stress-Strain-Time Data for Asphalt Concrete." *Trans. Soc. Rheology,* Vol. 8 (1964).

139. Secor, K. E., and Monismith, C. L., "Viscoelastic Response of Asphalt Paving Slabs Under Creep Loading." *Hwy. Res. Record No. 67* (1965) pp. 84-97.

140. Seed, H. B., Chan, C. K., and Lee, C. E., "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 611-636.

141. Seed, H. B., and Monismith, C. L., "Moderators' Summary Report of Papers Presented for Discussion at Session V." *Proc. Internat. Conf. on Structural Design of Asphaltic Pavements,* Univ. of Michigan (1962) pp. 551-580.

142. Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K., "Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests." *NCHRP Rep. 35* (1967) 117 pp.

143. *Shell 1963 Design Charts for Flexible Pavements.* Shell Internat. Petroleum Co., Ltd. (1963).

144. Shupe, J., and Taylor, D. C., "Field and Laboratory Investigations on Hot Mix Asphalt Concrete." *HRB Bull. 154* (1957) pp. 1-20.

145. Simpson, W. C., Griffin, R. L., and Miles, T. K., "Correlation of the Microfilm Durability Test with Field Hardening Observed in the Zaca-Wigmore Experimental Project." *Spec. Tech. Publ. No. 277,* ASTM (1959) pp. 52-63.

146. Skok, E. L., and Finn, F. N., "Theoretical Concepts Applied to Asphalt Concrete Pavement Design." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements,* Univ. of Michigan (1962) pp. 412-440.

147. Skog, J., and Zube, E., "New Test Methods for Studying the Effect of Water Action on Bituminous Mixtures." *Proc. Assn. Asphalt Paving Tech.,* Vol. 32 (1963) pp. 380-411.

148. Smith, T. L., "Stress-Strain-Time-Temperature Relationships for Polymers." *Spec. Tech. Publ. No. 325,* ASTM (1962).

149. Smith, V. R., "Triaxial Stability Method for Flexible Pavement Design." *Proc. Assn. Asphalt Paving Tech.,* Vol. 18 (1949) pp. 63-94.

150. Speer, T. L., "Laboratory Traffic Tests of Miniature Asphalt Highways; Second Progress Report." *Proc. Assn. Asphalt Paving Tech.,* Vol. 31 (1962) pp. 507-532.

151. Tons, E. and Krokosky, E. M., "Tensile Properties of Dense Graded Bituminous Concrete." *Proc. Assn. Asphalt Paving Tech.,* Vol. 32 (1963) pp. 497-529.

152. Traxler, R. H., "Durability of Asphalt Cements." *Proc. Assn. Asphalt Paving Tech.,* Vol. 32 (1963) pp. 44-63.

153. Traxler, R. H., and Schweyer, H. E., "Separating Asphalt Materials, *n*-Butanol-Acetone Method." *Oil and Gas Jour.* Vol. 57.

154. Trott, J. J., "An Apparatus for Recording the Duration of Various Temperatures in Roads." *Roads and Road Constr.* (Nov. 1963).

155. Troxell, G. E., and Davis, H. E., *"Composition and Properties of Concrete."* McGraw-Hill (1956).

156. van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumens and its Relation to Routine Test Data." *Jour. Appl. Chem.* (May 1954).

157. van der Poel, C., "Road Asphalt." In *Building Materials, Their Elasticity and Inelasticity.* M. Reiner, ed. Interscience (1954).

158. Vallerga, B. A., "On Asphalt Pavement Performance." *Proc. Assn. Asphalt Paving Tech.,* Vol. 24 (1955) pp. 79-102.

159. Vallerga, B. A., "The Effects of Aggregate Characteristics on the Stability of Asphaltic Paving Mixtures." Paper presented at 41st Ann. Conv., National Sand and Gravel Assn., Los Angeles (Feb. 13, 1957).
160. Vallerga, B. A., "Emulsified Petroleum Oils and Resins in Reconstituting Asphalts in Pavements." *Hwy. Res. Record No. 24* (1963) pp. 62-105.
161. Vallerga, B. A., et al., "Effect of Shape, Size and Surface Roughness of Aggregate Particles on the Strength of Granular Materials." *Spec. Tech. Publ. No. 212* (1957) pp. 63-76.
162. Vallerga, B. A., Monismith, C. L., and Granthem, K., "A Study of Some Factors Influencing the Weathering of Paving Asphalts." *Proc. Assn. Asphalt Paving Tech.*, Vol. 26 (1957) pp. 126-150.
163. Vesic, A. S., "Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements." *NCHRP Report 10* (1964).
164. Wada, Y., and Hirosi, H., "Glass Transition Phenomena and Rheological Properties of Petroleum Asphalt." *Jour. Phys. Soc. Japan*, Vol. 15, No. 10 (Oct. 1960).
165. Whiffin, A. C., and Lister, N. W., "The Application of Elastic Theory to Flexible Pavements." *Proc. Internat. Conf. on Structural Design of Asphalt Pavements*, Univ. of Michigan (1962) pp. 499-521.
166. White, O. A., "Service Behavior of Asphalt Concrete." *HRB Bull. 300* (1961) pp. 18-31.
167. Wilkins, E. B., and Campbell, G. D., "Flexible Pavement Design Based on Benkelman Beam Rebound Measurements." *Proc. Assn. Asphalt Paving Tech.*, Vol. 32 (1963) pp. 412-446.
168. Williams, S., and Lee, A., "Load-Deflection Study of Selected High-Type Flexible Pavement in Maryland." *HRB Bull. 177* (1958) pp. 1-20.
169. Zube, E., and Cechetini, J., "Expansion and Contraction of Asphalt Concrete Mixes." *Hwy. Res. Record No. 104* (1965) pp. 141-163.

APPENDIX

SUGGESTED EXPERIMENTAL PROCEDURE FOR A STUDY OF THE FATIGUE OF ASPHALTIC SURFACING

Chapter Nine outlines those factors believed by the authors to be of the highest priority as regards future research related to the factors involved in the design of asphaltic pavement surfaces. This appendix has been prepared as an illustrative guide to the implementation of the recommended research dealing with the fatigue of asphaltic surfacings.

It is emphasized that this suggested experiment represents the view of the authors only and is not intended to imply that alternate experiments are not possible. Similar types of experiments may be developed for durability and thermal stress. Experiment designs for these factors have been left to specific investigators, inasmuch as the experiments must be tailored to the project, funds, and time available.

EXPERIMENT DESCRIPTION

To establish the reliability of using laboratory fatigue measurements as a means of predicting the fatigue performance of in-service asphaltic surfacings, two investigative approaches are suggested: (a) Field testing of prototype pavement sections, or (b) field testing of in-service pavements.

The prototype sections offer the advantages of being able to investigate a range of selected variables or factors in tests which can be carried out over relatively short periods of time in a reasonably controlled climate environment.

The principal disadvantages of a prototype section are the somewhat unusual conditions used for testing, in which boundary conditions, methods of loading, and reduced overall time may influence the results.

The full-scale in-service method offers the self-evident advantage of being an actual pavement in an actual environment and subjected to actual loadings.

The full-scale method has the inherent disadvantage of requiring a long time to obtain results, with only limited control over the test variables.

Either type of investigation would be suitable; however, in order to evaluate specific factors such as aggregate gradation, surface texture, asphalt grade, void content, climate, thickness, and structural design factors (e.g., variations in strength parameters of underlying layers), the prototype pavement experiment would seem more appropriate. Eventually, in order to evaluate the overall validity of applying fatigue concepts to pavement design, a correlation with in-service pavements will be required.

FATIGUE RESEARCH WITH PROTOTYPE PAVEMENT

The following is a suggested outline for a research program to obtain answers as to the validity of fatigue relationships to performance from prototype tests:

1. *Design and construction.* The prototype facility should

be of sufficient size to be compacted with actual rollers of the type used on highway construction. In all probability the area should be 12 ft wide and at least 10 to 12 ft long for each test variable. A uniform subgrade material approximately 2 to 3 ft thick should be placed in accordance with the requirements of the experiment. The thicknesses of components, types of materials used, and placement requirements will also depend on the requirements of the experiment.

2. *Tests and measurements.* The following tests and measurements should be performed as a minimum; additional requirements will be dictated by the particular objectives of the experiment:
   (a) Subgrade.
      (1) Routine construction control test.
      (2) Identification and classification tests.
      (3) Measure elastic response; e.g., resilient modulus.
   (b) Untreated aggregates.
      (1) Routine construction control.
      (2) Identification tests.
      (3) Measure elastic response; e.g., resilient modulus.
   (c) Asphaltic concrete.
      (1) Routine mix design tests.
      (2) Routine construction control tests.
      (3) Identification and durability tests for asphalt.
      (4) In-place density and permeability.
      (5) Measure elastic response; e.g., stiffness, complex modulus, etc.
      (6) Laboratory fatigue properties using both laboratory- and field-compacted specimens.
      (7) Strain on the upper and lower surfaces of the asphaltic concrete.
      (8) Ambient and asphaltic concrete temperature.
3. *Simulated traffic loading.* A variety of methods can be used, depending on the degree of sophistication required, the objectives of the experiment, and the funds available. In general, it is not considered that actual moving wheel loads will be essential. The major requirement will be to apply dynamic loads which will compare with those expected from truck traffic. Initially the loads used should be selected to produce levels of strain similar to those used in the laboratory fatigue tests.
4. *Test variables and recommended priority.*
   (a) Evaluation of constant strain and constant stress fatigue tests and their applicability to asphaltic surfacings of various thicknesses. For this research both constant stress and constant strain fatigue tests will be required.
   (b) Evaluation of the applicability of the stiffness modulus to fatigue life. This program should be carried out in phases, starting initially with a single, essentially dense-graded aggregate tested with various asphalt grades and void contents.
   (c) Evaluation of compound loading conditions on fatigue life. In this effort it will be necessary to program random loadings, of either the repeated-block or pseudo-random type described by Deacon.

To maintain adequate control over climate conditions, the prototype tests should be performed within some type of enclosure.

FATIGUE RESEARCH USING IN-SERVICE PAVEMENTS

In general, the objectives of this type of research are the same as for the prototype tests previously outlined. The investigation could procede somewhat as follows:

1. Select new constructions with a range of surfacing thicknesses, material properties, and environments.
2. Conduct construction and material testing similar to that specified for prototype model studies.
3. Obtain or develop continuous recordings of climatological conditions.
4. Make frequent traffic counts and loadometer surveys.
5. Make seasonal condition surveys of pavement performance.

The general approach described herein could lead to the type of definitive information required in order to incorporate fatigue properties of asphaltic concrete into design standards and material requirements.

As mentioned in the introductory remarks of this appendix, experiment designs for durability can be programmed similar to those outlined for fatigue. The main objective of the durability studies should be to develop reliable information regarding the long-term performance of asphaltic surfacings subjected to a wide variety of environments. It would appear that this research should be divided into two aspects; namely, (a) durability in terms of structural performance, and (b) durability in terms of cementing properties (adhesion) as a function of the properties of the asphalt-aggregate system. In all probability, in-service highways would provide the most logical test area. Small experiments incorporating materials with various properties could be included within normal construction projects.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue
Washngton, D.C. 20418

Rep. No.	*Title*
—*	A Critical Review of Literature Treating Methods of Identifying Aggregates Subject to Destructive Volume Change When Frozen in Concrete and a Proposed Program of Research—Intermediate Report (Proj. 4-3(2)), 81 p., $1.80
1	Evaluation of Methods of Replacement of Deteriorated Concrete in Structures (Proj. 6-8), 56 p., $2.80
2	An Introduction to Guidelines for Satellite Studies of Pavement Performance (Proj. 1-1), 19 p., $1.80
2A	Guidelines for Satellite Studies of Pavement Performance, 85 p.+9 figs., 26 tables, 4 app., $3.00
3	Improved Criteria for Traffic Signals at Individual Intersections—Interim Report (Proj. 3-5), 36 p., $1.60
4	Non-Chemical Methods of Snow and Ice Control on Highway Structures (Proj. 6-2), 74 p., $3.20
5	Effects of Different Methods of Stockpiling Aggregates—Interim Report (Proj. 10-3), 48 p., $2.00
6	Means of Locating and Communicating with Disabled Vehicles—Interim Report (Proj. 3-4), 56 p. $3.20
7	Comparison of Different Methods of Measuring Pavement Condition—Interim Report (Proj. 1-2), 29 p., $1.80
8	Synthetic Aggregates for Highway Construction (Proj. 4-4), 13 p., $1.00
9	Traffic Surveillance and Means of Communicating with Drivers—Interim Report (Proj. 3-2), 28 p., $1.60
10	Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements (Proj. 1-4), 31 p., $2.80
11	Effect of Control Devices on Traffic Operations—Interim Report (Proj. 3-6), 107 p., $5.80
12	Identification of Aggregates Causing Poor Concrete Performance When Frozen—Interim Report (Proj. 4-3(1)), 47 p., $3.00
13	Running Cost of Motor Vehicles as Affected by Highway Design—Interim Report (Proj. 2-5), 43 p., $2.80
14	Density and Moisture Content Measurements by Nuclear Methods—Interim Report (Proj. 10-5), 32 p., $3.00
15	Identification of Concrete Aggregates Exhibiting Frost Susceptibility—Interim Report (Proj. 4-3(2)), 66 p., $4.00
16	Protective Coatings to Prevent Deterioration of Concrete by Deicing Chemicals (Proj. 6-3), 21 p., $1.60
17	Development of Guidelines for Practical and Realistic Construction Specifications (Proj. 10-1,) 109 p., $6.00
18	Community Consequences of Highway Improvement (Proj. 2-2), 37 p., $2.80
19	Economical and Effective Deicing Agents for Use on Highway Structures (Proj. 6-1), 19 p., $1.20
20	Economic Study of Roadway Lighting (Proj. 5-4), 77 p., $3.20
21	Detecting Variations in Load-Carrying Capacity of Flexible Pavements (Proj. 1-5), 30 p., $1.40
22	Factors Influencing Flexible Pavement Performance (Proj. 1-3(2)), 69 p., $2.60
23	Methods for Reducing Corrosion of Reinforcing Steel (Proj. 6-4), 22 p., $1.40
24	Urban Travel Patterns for Airports, Shopping Centers, and Industrial Plants (Proj. 7-1), 116 p., $5.20
25	Potential Uses of Sonic and Ultrasonic Devices in Highway Construction (Proj. 10-7), 48 p., $2.00
26	Development of Uniform Procedures for Establishing Construction Equipment Rental Rates (Proj. 13-1), 33 p., $1.60
27	Physical Factors Influencing Resistance of Concrete to Deicing Agents (Proj. 6-5), 41 p., $2.00
28	Surveillance Methods and Ways and Means of Communicating with Drivers (Proj. 3-2), 66 p., $2.60
29	Digital-Computer-Controlled Traffic Signal System for a Small City (Proj. 3-2), 82 p., $4.00
30	Extension of AASHO Road Test Performance Concepts (Proj. 1-4(2)), 33 p., $1.60
31	A Review of Transportation Aspects of Land-Use Control (Proj. 8-5), 41 p., $2.00
32	Improved Criteria for Traffic Signals at Individual Intersections (Proj. 3-5), 134 p., $5.00
33	Values of Time Savings of Commercial Vehicles (Proj. 2-4), 74 p., $3.60
34	Evaluation of Construction Control Procedures—Interim Report (Proj. 10-2), 117 p., $5.00
35	Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests (Proj. 1-3(3)), 117 p., $5.00
36	Highway Guardrails—A Review of Current Practice (Proj. 15-1), 33 p., $1.60
37	Tentative Skid-Resistance Requirements for Main Rural Highways (Proj. 1-7), 80 p., $3.60
38	Evaluation of Pavement Joint and Crack Sealing Materials and Practices (Proj. 9-3), 40 p., $2.00
39	Factors Involved in the Design of Asphaltic Pavement Surfaces (Proj. 1-8), 112 p., $5.00

* Highway Research Board Special Report 80.

THE NATIONAL ACADEMY OF SCIENCES is a private, honorary organization of more than 700 scientists and engineers elected on the basis of outstanding contributions to knowledge. Established by a Congressional Act of Incorporation signed by President Abraham Lincoln on March 3, 1863, and supported by private and public funds, the Academy works to further science and its use for the general welfare by bringing together the most qualified individuals to deal with scientific and technological problems of broad significance.

Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

THE NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

THE NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U. S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

THE DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

THE HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering, is a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.