# HIGHWAY RESEARCH RECORD

## Number 273

## Bituminous Materials and Mixes

9 Reports

Subject Area

31 Bituminous Materials and Mixes
32 Cement and Concrete

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Washington, D. C., 1969 Publication 1650

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# Foreword

Papers in this RECORD cover a wide range of subjects in bituminous paving. Of special interest to those attempting to understand the fundamental flow characteristics of bituminous paving mixtures are the papers by Schweyer and Busot, Yackovlev and Barenberg, and Majidzadeh.

Schweyer and Busot advance the view that for rheological purposes the behavior of bituminous paving mixtures can be defined as (a) the stress-strain relation of a zero rate of strain; (b) a measurement of consistency related to classical viscosity; and (c) a "characteristic" time that is a measure of the retention of previously induced strains. Yackovlev and Barenberg, however, conducted rheological studies using the complex modulus as a definitive measurement of viscoelasticity. A static loading condition was used to predict the response under dynamic loading. Where the two preceding papers dealt with fresh mixes, the paper by Majidzadeh deals with rheological behavior as influenced by aging. Of special interest is the implication that the aging susceptibility of the binder and aging susceptibility of the mixtures are correlatable with a measure of viscosity. Majidzadeh also found appreciable difference in the rheological response of various asphalts to aging.

This RECORD also includes several papers of interest to designers of bituminous paving mixtures. Gallaway and Hargett advance a method for combining aggregates of widely different specific gravities to meet a conventional gradation specification. The blending procedure utilizes a volumetric analysis and will yield gradations similar in particle content to conventionally graded aggregates.

Lee found that the Marshall stability and flow obtained on core specimens offers little information in evaluation of the stability and plastic resistance of an in-place paving mixture. He further notes that briquets remolded from the pavement mixture provide more informative data than tests on cores. A quantitative measure of particle orientation is advanced by Lees and Salehi. The procedure and the rationale behind it are explained.

A rapid determination of asphalt content was evaluated by Antrim and Busching. Good accuracy is reported with a test time of approximately 25 minutes. The limitations are the need for calibration tests to account for different operators and aggregate types.

A general summary of the work performed during the past 15 years on the results of adding natural rubber to asphalt is presented by Thompson. The paper does not present any new information but does provide an overview of the subject.

A dynamic loading procedure was developed by Majidzadeh and Stander to investigate the stripping that occurred in a sand-asphalt layer placed on US 231 in Florida. After the authors found that the immersion compression test did not correlate with the field phenomena, a dynamic test procedure was investigated. They found that dynamic preconditioning of soaked sand asphalt briquets did reveal a loss of compressive strength and thereby concluded that the dynamic procedure showed promise of becoming a useful tool to assess moisture effects.

—Jack H. Dillard

# Contents

# A New Approach in Asphalt Rheology

HERBERT E. SCHWEYER and J. CARLOS BUSOT, Department of Chemical Engineering, University of Florida

This paper presents arguments for the need of using rheology in specifications for paving asphalt. In particular, the paper discusses the need for determining the deformation characteristics employing asphalt invariant parameters with the objective that such results will be useful in bridging the gap from the scientific measurement of asphalt to the utilization in the design of better flexible pavements.

A review of recent publications pertinent to the subject is included and refers in particular to the more sophisticated considerations of rheology phenomenological behavior. From this background, a new method of characterizing the rheology of asphalt is proposed as being more rigorous and, it is hoped, more useful for design purposes.

The procedure described uses the Instron testing machine with an environmental chamber and a compression test in which the basic concept of compressibility is utilized for evaluating three rheological parameters. These parameters are the stress-strain relation at a zero rate of strain condition, a measurement of consistency related to classical viscosity, and a characteristic time, which is a measure of the memory of previous strains that the material retains and that influence its rheological behavior. These three independent material parameters are sufficient to describe the asphalt for all rheological purposes. Line diagrams are presented to illustrate the procedure and an example of the proposed analysis for an 85 to 100 penetration asphalt is given.

•AS AN introduction, we wish to emphasize that this paper presents mainly new concepts for asphalt rheology with only limited data. Its purpose is to reiterate, for the record, the need for new approaches and to promote discussions and suggestions in using compression techniques.

## NEED FOR ASPHALT RHEOLOGY

The flow properties of asphalt have been measured in a variety of ways from the time it was first used as an engineering material. Originally these methods of measuring rheology were empirical in nature; examples are the penetration test, ductility test and softening point, which subsequently were studied from theoretical aspects and were found to have serious limitations. It is necessary to study the flow properties of asphalts in order to have some measure of consistency and uniformity of materials being used in a given application.

Consideration of this has led to emphasis on studying flow properties using invariant parameters. Although much progress has been made, methods used today are still inadequate for the intended purposes. In fact, it may be unequivocally stated that no specification test for asphalt, as measured today, can be employed directly for the

Paper sponsored by Committee on Characteristics of Bituminous Materials and presented at the 48th Annual Meeting.

design of bituminous pavements. Thus, there is a great need for a suitable test that can be run by technicians and that can be interpreted correctly in terms of true rheological phenomena and be of immediate application to design.

The review of literature will primarily refer to certain early articles and also to more recent publications that contain bibliographies from which one may obtain a more complete history. The basic concept in this paper relates to thermodynamic considerations that Mack (1, 2, 3) was one of the first to recognize. Other investigators have studied, with varying degrees of success, the application of physicochemical principles to asphalts. Perhaps one of the most important developments is the application of the superposition principle to study the rheological behavior of asphalt over a wide temperature range. Shoor, Majidzadeh and Schweyer (4) present an illustration of this along with certain other applications of newer techniques. Their work is by no means novel, but it does include a number of different types of asphalts for study and the references may be consulted for other investigations in the field. Certain specific references selected by us as being quite pertinent to the studies of the nature of asphalt deformation over a wide range of temperature include Heukelom (5), who discussed the experimental aspects of certain deformation tests; Schmidt and Santucci (6); Sisko (7); Majidzadeh and Schweyer (8); and Moavenzadeh (9). These men as well as others have worked with asphalts in those temperatures and conditions which produce asphalt fracture. Krchma (10) proposes a new type of softening point as a measure of the rheological characteristics of asphalts in the intermediate temperature range, where the temperature at which the asphalt demonstrates certain rheological properties is considered a critical characteristic.

These new approaches have been presented during approximately the last ten years and include the works of Gaskins, et al (11) and Brodnyan (12), who have provided certain methods to approach the asphalt rheology problem. The work here will be based on this background. The basic rheology concepts and rational mechanics approach used can be found in Slattery (13), Coleman, Markovitz, and Noll (14), Middleman (15), Truesdale (16, 17), and Lodge (18).

## CONCEPT OF APPLIED RHEOLOGY AND SPECIFICATIONS

The ultimate goal of rheological studies on paving asphalts is to apply the results to specifications. There are really two types of specification requirements for these materials, and they must be kept in proper perspective. First, it is necessary to know the requirements for asphalt material to behave properly under selected process conditions. This is a process specification. An example is the consideration of deformations in pavements under compaction as influenced by the viscosity of the asphalt. Second, there are material responses in the form of rheological behavior that are a function of the composition of the asphalt. This is a product specification.

In general the procedures for testing flow characteristics of asphalts above 122 F present no problem because paving asphalt cements of 85 plus penetration are essentially Newtonian. These results provide numerical values for viscosity (assuming no complications due to hardness) that can be compared with required specifications for the process conditions—mixing and compaction requirements. However, as lower temperatures are required for consideration in either the process or product specification categories, it becomes necessary to consider more complicated methods of measurement to obtain significant numerical values. These test temperatures may range down to 0 F and, accordingly, special equipment is required. As noted, work along these lines has been done and the resulting development of highly sophisticated equipment permits considerably more application of fundamental rheological principles to analysis of the data. Furthermore, the ultimate use of such data for translation into highway pavement design is a very necessary goal for improved understanding of the rheological behavior of flexible pavements.

Majidzadeh (19) has indicated a direction in which the necessary data on rheology of the asphalt component may be used for rational design of highway pavements in connection with sand-asphalt mixtures where three design criteria are noted—stability, durability, and resistance to fatigue. However, as far as is known to us, the rheological values have never been translated into practical applications for the design of bituminous

Figure 1. Instron floor model.

paving mixtures. It would appear that the reason for this lack of use of such information is that to date all rheological data are not presented as invariant parameters. Such data cannot be used because the data presented have as yet not considered the basic constitutive assumptions defining the phenomenological behavior of the asphalts. An understanding of these relationships will provide the bridge between specifications and the rational design of bituminous pavements.

## A SUGGESTED NEW APPROACH IN ASPHALT RHEOLOGY

Researchers at the Asphalt Laboratory at the University of Florida have, for a number of years, studied the rheological behavior of bituminous materials. The initial investigations began with the sliding plate viscometer developed by Shell Oil Company. This was followed by the cone and plate viscometer developed by Sisko of the American Oil Company. Concurrently, work was carried out using low-temperature creep tests on asphalt, and initial studies were started using an Instron rheometer. Because of the complicated nature of the viscoelastic flow of asphalt cements, it soon became apparent that all of these instruments had certain limita-

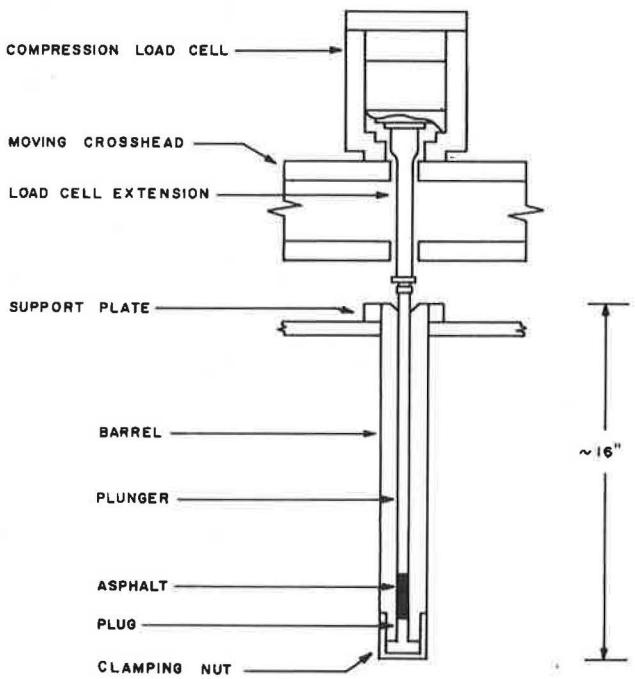


Figure 2. Instron rheometer (modified).

Figure 3. Rheometer assembly.

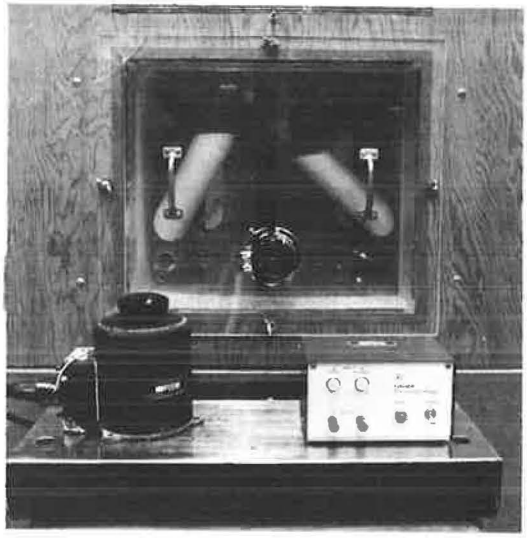

Figure 4. Environmental chamber.

tions. As a result, attention was focused on using the Instron testing equipment in order to measure a time constant characteristic of the flow of asphalt at temperatures of 32 F (0 C). Study of the stress relaxation indicated a possible approach for measurement of this characteristic time in addition to evaluating a viscosity parameter. For this purpose an Instron machine was used (Fig. 1). An Instron rheometer with a plug replacing the orifice at the bottom (Fig. 2) was utilized to study stress relaxation. The components of the barrel assembly are shown in Figure 3, and the environmental chamber for the testing machine is shown in Figure 4.

## COMPRESSION OF ASPHALT

The compression of asphalt in a confined space as described here is new, as far as known. Certain work is being carried out in the University of Florida laboratory to evaluate the coefficient of compressibility at different temperatures. However, there are certain considerations in such experimental work that must be recognized. For example, the compression load under strain may be studied with the drag on the wall being included. This requires evaluation after an equilibrium stress value is attained at a given strain, which extends the time of an experiment. Conversely, the walls of the confining barrel may be lubricated, permitting study of "internal" viscosity and faster determination of equilibrium data. Each method has value in providing information concerning the rheology of asphalt, as will be demonstrated with examples and preliminary consideration of the significance of the results.

### Deformation With Drag

If an asphalt is placed in a confined geometry and subjected to deformation, a curve such as shown in Figure 5 is obtained. If the deformation is stopped, a stress relaxation will occur, declining to some equilibrium stress value denoted by $\tau_0$. (This may be considered the value obtained at a zero deformation rate.) However, the machine also deforms as shown in Figure 5 and, being essentially elastic, shows a time deformation curve with only a very small relaxation of stress. The infinite time asymptote designating $\tau_0$ assumes there is no residual stress $\tau_r$ above $\tau_0$. For the real case there may be a definite residual stress $\tau_r$, which may or may not be significantly higher than the equilibrium value.

The relations for the deformations are shown in an exaggerated scale in Figure 6, where the difference between the total deformation $\Delta R$ and the machine deformation $\Delta M$ represents the deformation in the sample, $\Delta S$. In a cylindrical section it is assumed that radial changes in the machine barrel are not significant. An illustration of the time-deformation curves is shown in Figure 7, which also shows the relationships involved.

In the actual performance of a test the samples are preheated in a tared container and poured into the bottom of the heated barrel to a predetermined length of 1 to 2 inches. The plug is inserted and capped and the barrel assembly placed in the environ-

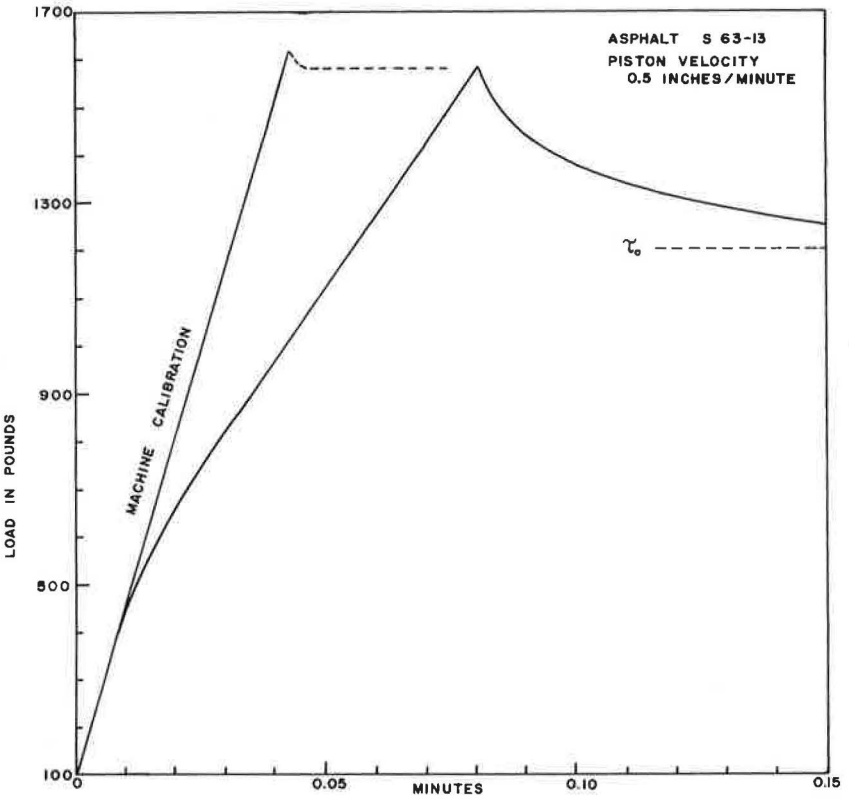


Figure 5. Deformation curve with drag.

mental chamber. The exact amount of asphalt is determined from the residual weight of the tared container and contents.

After the assembly has been cooled to the test temperature, an initial small set stress is applied to remove backlash in the assembly. Application of stress after the temperature has been lowered below 10 C virtually eliminates leakage of the sealing O-rings. Checks have been carried out by determining the volume of asphalt before and after conducting the experiments. Initial and final volumes check within ±0.03 percent, which is approximately 1 percent of the measured deformation during experiments. Starting from this point the machine crosshead is run at different deformation rates and times, and the stress is recorded. The sample is then held and stress relaxation permitted to approach an equilibrium value. Different crosshead velocities are used on the same sample by permitting the sample to rebound and by repeating the deforming procedure. It should be obvious that a given amount of strain will require less time at a higher crosshead speed, but these can be normalized for a given

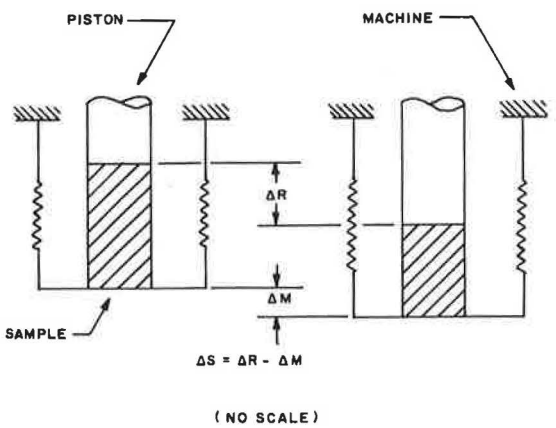


Figure 6. Model for compression (no scale).

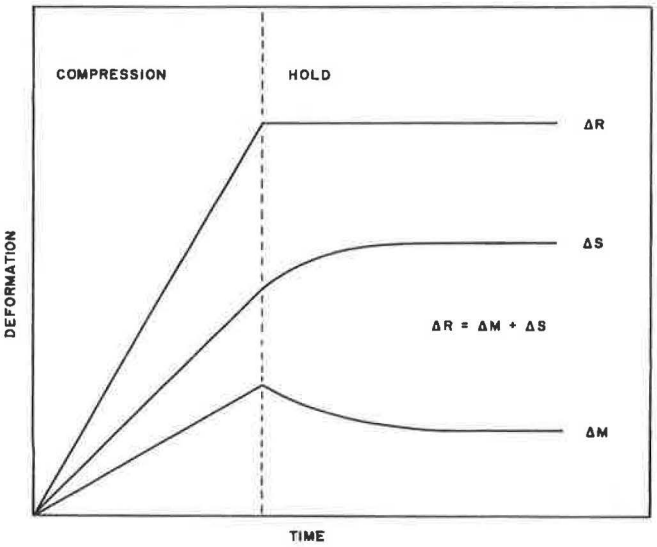


Figure 7. Comparison of strains.

amount of strain as shown in Figure 8. These are corrected strains obtained from the relationships of Figure 5. The machine calibration curves (stress vs strain is the same regardless of the crosshead velocity) are superimposed on the actual experimental recordings. At a given stress the strain is determined as $\Delta S = \Delta R - \Delta M$, the data plotted in Figure 8. The different strains are normalized from the time-deformation curves recorded by the machine.

The zero-rate curve shown in Figure 8 is for the equilibrium values discussed and represents the thermodynamic reversible change in density with pressure at constant temperature 32 F. The other curves represent the real situation, and since the deformation coordinate is related to density, the slopes represent the dynamic expressions for change of stress with respect to density of $(\partial\tau/\partial\rho)_T$, which is related to internal cohesive forces.

As noted in Figure 8 on asphalt S63-13, during deformation at 32 F there is a drag effect where the asphalt is in contact with the walls. Thus, excess stresses above the zero rate may be utilized to evaluate this viscous drag effect. Theoretically, the zero-rate (or reversible) curve is a measure of the elasticity of the material at a given temperature. It can be determined from procedures shown in Figure 5.

The excess stress for different rates of deformation is not constant (parallel to the zero-rate line). This could be due to a pressure-dependence of the material parameters as well as other possible nonlinear phenomena. From other work carried out in this laboratory there is evidence that the measured consistency (viscosity) may be influenced by changes in density. Figure 8 shows that pressures up to 15,000 psi are approached and these could definitely affect the rheology of the material.

In addition, part of the deviation of the curves from the zero rate in Figure 8 may be caused by memory effects. The history of deformations may determine the state of stress at the time of consideration. The memory (influence of past deformations) of real materials is a fading memory. This means that a deformation that occurred a long time in the past will have little influence on the present deformation regardless of the magnitude of the earlier deformation. The converse is true for materials that have a permanent memory.

A perfectly elastic material will permanently remember only its unstressed state ($\tau$ is unique function of strain from a reference state). Conversely, a Stokesian fluid will have such a short memory that only the deformation instantly before the present

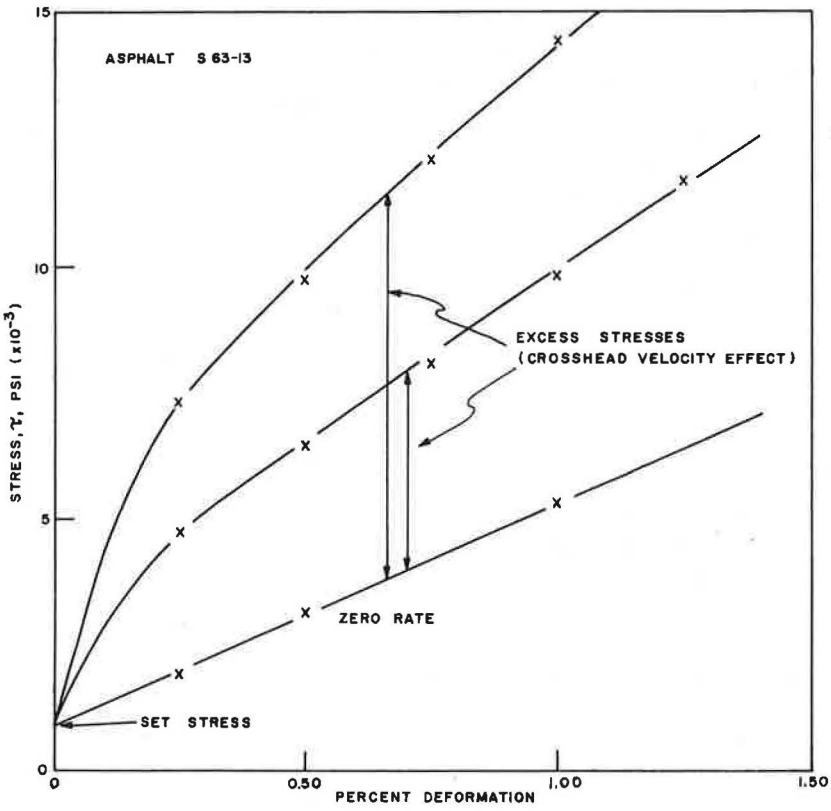


Figure 8. Normalized and corrected data for asphalt cement S63-13 with drag.

will have any influence ($\tau$ is a function of rate of strain). The effect of a given deformation in the past may thus be characterized by a time. This time is usually called the characteristic time and an interesting discussion of it has been presented by Slattery (13). Slattery explains the different viewpoints on characteristic times by Truesdale (17). Essentially, Slattery's contention is that differentiation between the contribution of the material and the contribution of the process itself to the overall time-dependence is difficult and somewhat arbitrary. Determination of different time constants when testing different materials under the same experimental conditions is enough to warrant assignment of these time constants to the materials.

[A Stokesian fluid is one where the stress caused by deformation is only a function of the rate of deformation; it may be Newtonian or non-Newtonian (14). Asphaltic cements used in paving have memories at ambient temperatures that will be between the limits for these two extremes because they are viscoelastic. These memory effects are indicated by the experimental curves of Figure 7.]

Conceptually, the relationships discussed for the dynamic stress at essentially a constant rate of deformation might be considered as follows: The time-varying stress resulting from the application of a constant rate of deformation will be primarily determined by the consistency of the material. Mathematically this dynamic stress is the product of three factors. It should be emphasized that this equation is of no rheological value unless K can be described at each material point and time.

$$\tau = MU \cdot K \cdot g(\text{memory}) \qquad (1)$$

where

$\tau$ = dynamic stress above that for zero-rate deformation, dynes/cm$^2$;
MU = consistency modulus, poises;
K = rate of deformation = f(rate of compression and geometry); and
g(memory) = dimensionless time-dependent factor representing the memory of the material.

For the special cases of perfectly elastic material and Stokesian fluids, g = 0, and g = a constant independent of time, respectively.

## Deformation With Lubrication

To eliminate the drag effect, the barrel of the cylinder and the plunger were coated with a thin film of silicone lubricant and compression tests carried out as described previously at 32 F. With the drag on the walls eliminated, the deformation during compression is homogeneous, allowing an invariant evaluation of the phenomenon. The parameters thus obtained when studying the response to different deformation histories can be evaluated without influence of different geometries and machine characteristics. A homogeneous compression occurs when three material particles remain colinear throughout the deformation. A simple check on the degree of homogeneity of the deformation is to deform different volumes of sample ($V_1$, $V_2$, etc.) and adjust the crosshead speed (K) such that

$$\frac{K}{V_1} = \frac{K}{V_2}$$

The response to these experiments should be identical.

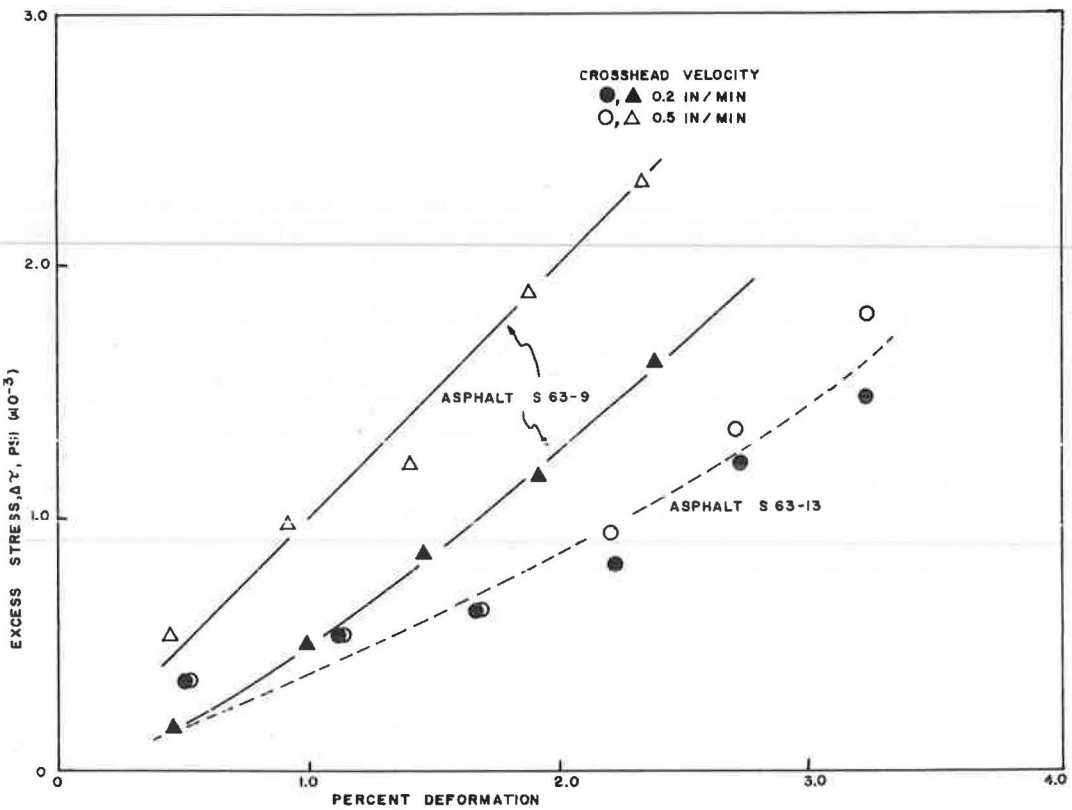


Figure 9. Comparative data on two asphalt cements with lubrication.

The results on two asphalts shown in Figure 9 present certain interesting information. Asphalt cement S63-9 is from a Texas source and it shows considerable difference in the deformation curves at two different rates of deformation. If the material were completely elastic, the stress developed would be essentially independent of the rate of deformation. This is approximated by the preliminary data for asphalt S63-13, which is an air-blown asphalt. Such materials are known to exhibit elastic properties with a decrease in viscosity at high rates of deformation.

In the case of asphalt S63-9, the excess stress above that for the zero deformation rate is a measure of visco-compression effects, which at this time have not been analyzed completely. However, the significant differences for the two asphalts in Figure 9 are considered important. This is particularly true because other information from work in this laboratory has indicated appreciable differences in the changes of their internal energies (u) with volume (v). This has been studied by Ronk, Busot, and Schweyer (20) in measuring the coefficients of thermal expansivity and isothermal compressibility. The value of $(\partial u/\partial v)_T$ can be evaluated, and it is an indication of the cohesive forces for these materials.

The basis for further consideration of predictions and correlations of the properties discussed may be found in Bondi (21) and Middleman (15). These references review most of the pertinent theories and correlation methods. However, the experimental observations reported in this paper cannot be explained rigorously by present theories. Most viscosity theories are limited by the assumption of incompressibility, or by considering deformations at constant density (isochoric shear and elongational experiments). Free-volume theories on the other hand have failed to rigorously explain the low-temperature viscosity data of certain liquids (22). Moreover, asphalts are complex mixtures requiring perhaps different and sophisticated mixture theories for each property considered.

## SIGNIFICANCE OF RESULTS

Although considerable experimental data have been obtained, no quantitative measurements of deformation parameters are given for the preliminary homogeneous compressive deformation reported here. The significance of these results has not been completely developed as yet, and until this is finished it would appear that discretion should be employed before drawing conclusions. The difference in response between the "homogeneous" and "drag" compression tests will not be interpreted, but instead Table 1, giving empirical characteristic time, consistency, and compressibility, is presented.

The stress decay curves (Fig. 5) were used to evaluate a characteristic time. The ratio of the initial slope $(d\tau/dt)_{t=0}$ to the initial value of the rate of change of this slope $(d^2\tau/dt^2)_{t=0}$ has the dimension of time. This arbitrary time, when obtained at exactly the same experimental condition for different asphalts, is assigned as their characteristic time, $\theta$. This is only an empirical indication of how long the asphalts are able to remember past deformations. Similarly, the excess stress at 2 percent

TABLE 1

PRELIMINARY EVALUATION OF MATERIAL RHEOLOGICAL PARAMETERS AT 32 F

| Identification | Penetration 77 F | Viscosity 140 F, poises | Compressibility, percent strain/psi ($\times 10^4$) | Consistency $\mu$, poises ($\times 10^{-12}$) | Characteristic time, $\theta$, sec |
|---|---|---|---|---|---|
| Texas, SR Int. S63-9 | 85 | 1704 | 3.10 | 1.70 | 48 |
| AB. Naphthenic S63-13 | 89 | 1726 | 3.74 | 0.74 | 32 |
| Gulf Coast Naphthenic, SR S63-12 | 85 | 1368 | 3.10 | 2.53 | 84 |
| Los Angeles basin S63-20 | 89 | 1109 | 2.85 | 2.66 | 82 |

deformation divided by a specific rate of deformation gives a measurement of consistency $\mu$. The elastic compressibility was obtained from the stress-strain relation at zero rate of strain.

Many experiments have been carried out in the area of this research, but it is not the purpose here to present all the results because much of the exploratory work on repeatability, reproducibility, etc., will be discarded when procedures have become firm.

This information is presented in a preliminary fashion to indicate a new approach that might be taken for analysis of the rheological properties of asphalt. It represents an effort to obtain a better understanding in bridging the gap between the knowledge of the constitutive equations that define the material and utilization of this information in the actual design of a flexible pavement. The procedures and results as presented here are not considered final but are considered of sufficient interest to paving technologists to warrant presentation at this time and to solicit criticism and suggestions for additional work. This work is continuing and it is hoped that future information can be presented that will be more informative for application to design.

## ACKNOWLEDGMENT

The assistance of Donald Ronk in preparing the drawings and certain aspects of the experimental work in this research is gratefully acknowledged.

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# Some Aspects of the Rheological Behavior of Bituminous Mixes

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Rheological behavior of bituminous mixtures was studied and evaluated in terms of the theory of linear viscoelasticity. The material studied was tested under static loading conditions and its behavior under dynamic load was predicted by using the linear viscoelasticity theory.

Literature was reviewed and summarized. Tests were conducted under flexural and tensile stress conditions at three temperatures: 40, 70, and 100 F. Results were evaluated in terms of the complex modulus of elasticity, determined from curvature measurements of flexure specimens under creep-strain loading.

General trends for the complex modulus were established for bituminous mixes under the influence of temperature, applied load, curing time after compaction, and binder content of the mix.

In predicting the dynamic behavior of these materials from static tests by means of the complex modulus, it was found that such a prediction was significantly affected by the magnitude of the initial strain value in the creep-strain curve. The total response of the material was found to be mainly viscous in nature. As expected, temperature had a significant effect on the strength of the material and also affected critically the relationship between curvature and applied load.

•BASED on scientific tradition established over the past century and a half it is usually assumed that solids are elastic and obey certain laws, while liquids are viscous and obey a different set of rules. The difficulty with this approach is that each time a material deviates from the idealized properties of either extreme a new set of rules has to be established (13). The problem is compounded by the fact that, while some materials may be characterized through their mechanical properties in a relatively simple manner, characterization of most materials used in modern technology represents a challenge. This is because most materials fit neither of the two idealized classes of materials mentioned. The increasing use of polymers, or combined materials such as the soil-asphalt mixtures with which this study is concerned, makes it increasingly important to explore the response of these materials to applied stresses in light of their true behavior.

A material that has a partially elastic and partially viscous response is referred to as a viscoelastic material. Many viscoelastic materials exhibit nonlinear viscoelastic properties. Nevertheless, their behavior can be explained to a useful degree of approximation by the theory of linear viscoelasticity (12). This investigation concerned the evaluation of rheological behavior of bituminous mixtures under static creep-strain conditions. The complex modulus of elasticity was used with two objectives in mind: (a) as a means to evaluate the experimental findings developed in this study, and (b) as

Paper sponsored by Committee on Mechanical Properties of Bituminous Paving Mixtures.

a tool in evaluating the probable response of bituminous mixtures under dynamic load conditions. In the latter, the complex modulus of elasticity was used in transferring results from static creep-strain tests from the time domain to the frequency domain.

The main objectives of the study were:

1. To study the behavior of bituminous mixtures and evaluate that behavior in terms of the complex modulus concept of the theory of linear viscoelasticity;
2. To determine the influence of selected factors such as temperature, curing time after compaction, applied load, and binder content on the rheological behavior of such mixtures; and
3. To establish trends in the behavior of bituminous mixtures as affected by the factors studied in the course of this investigation.

Because of the extreme variability of the materials involved, the scope of this study is necessarily limited. The applicability of the results is limited to the materials used in this study, and consequently these results cannot be applied indiscriminately. However, the general trends established can be extended to similar materials.

## SOIL-ASPHALT MIXTURES AS VISCOELASTIC MATERIALS

The ability of the linear viscoelastic analysis to describe the behavior of bituminous mixtures has been a point of disagreement among investigators in the field. The temptation to use empirical results based on certain laboratory or field tests has been overwhelming in the past. The problem with this type of approach is that it encompasses in one or several parameters an extremely complex system of stresses, further complicated by the presence of several variables.

In view of these circumstances, several investigators have turned their attention toward a more fundamental approach in an attempt to reach an understanding of the true behavior of the materials under specific stress conditions, and to isolate the effect that the many variables might have on it. Such an approach has been used on flexible pavement systems in general, as well as on different types of asphaltic mixtures including soil-asphalt mixtures.

Although Skok and Finn (21) used an elastic analysis for asphaltic concrete pavement, they pointed out that the viscoelastic analysis should be the basic tool in examining the behavior of asphaltic mixtures once the mathematical difficulties of that approach had been streamlined. The theory of viscoelasticity is the logical tool for analyzing the materials that fall between the elastic and viscous extremes, and can also be a factor in simplifying the mathematical problems in analyzing pavements with viscoelastic materials. Baker (3), concurring that the rheological concepts developed through the theory of viscoelasticity permitted a more accurate representation of properties, advocated the use of those properties calculated by using the rheological concepts in an elastic theory of pavement analysis. Monismith and Secor (14) conclude that, at least for slow rates of loading, it appears that viscoelastic theory may be required to predict the behavior of asphalt concrete pavements. Wood and Goetz (24) found that sheet asphalt mixtures obeyed the laws of linear viscoelasticity for limited stresses and small deformations.

The majority of the investigators seem to agree that the viscoelastic approach toward explaining the behavior of soil-aggregate mixtures stabilized by means of a viscous binder is the most promising. Nevertheless, some investigators feel that an elastic analysis and predictions of behavior based solely on elastic theories, or modification thereof, offer a sufficiently good approximation.

## FACTORS INFLUENCING BEHAVIOR OF SOIL-ASPHALT MIXES

The rheological properties of bitumen-mineral aggregate compositions are highly temperature-dependent. Many investigators have turned their attention lately to the significant task of evaluating these rheological properties. In one such study, Pagen (16) used the complex modulus concept presented by Papazian (17) to evaluate the rheological response of bituminous concrete under dynamic loading. The concept of the

complex modulus used in the course of this investigation will be covered in detail in the next section of this paper. Because of the apparent success of this approach, it was selected for use in this investigation to evaluate and predict the rheological behavior of bituminous mixtures under static and dynamic loading conditions.

Abdel-Hady and Herrin (1), in a study made on compacted soil-asphalt mixtures with a medium-curing liquid asphalt as a binder, provided additional proof of the susceptibility of these materials to the rate of loading being applied. In the same study the importance of curing time after compaction, as shown by its influence on the rheological properties of the material, was established.

Most of these studies were performed with the material in compression or under more complex stress conditions such as those occurring in flexible pavement slabs. Tons and Krokosky (23) studied certain properties of bituminous concrete in tension. From this study the effects of temperature and loading time on the mixes were determined for this state of stress.

## Complex Modulus Concept

The complex modulus concept is based on the steady-state response of a linear viscoelastic material when subjected to sinusoidal stress (6). When such a material is placed under an alternating stress of a given frequency, its response will be a sinusoidal strain of the same frequency that lags the stress by a phase angle $\phi$.

If the applied stress is separated into two components, one in phase with the strain and the other 90 deg out of phase, then the ratio of the amplitude of the in-phase component of the stress to the amplitude of the stress to the amplitude of the strain represents the real part of the complex modulus. The ratio of the amplitude of the 90 deg out-of-phase component of the stress to the amplitude of the strain represents the imaginary part of the complex modulus (10).

Since no mention has been made of the type of stresses applied, it should be understood that the idea of the complex modulus M* is completely general. The real and imaginary parts, M′ and M″, of the complex modulus M* may be added vectorially on a complex plane to give $M^* = M' + iM''$. The absolute value of M*, which is a member of the set R, is given as the ratio of the amplitude of the stress to the amplitude of the strain. Analogous definitions can also be given. For example, the complex modulus of elasticity E*, the complex shear modulus G*, and the complex bulk modulus K* can appropriately be described by the stress and strain relationships. The work in this investigation will be concerned only with the complex modulus of elasticity E*.

Investigators concerned with the viscoelastic behavior of materials have used a variety of methods to obtain the complex modulus. These methods vary from the elegant method of Philippoff (19, 20) for measuring deformation in shear, through the simple flexure device of Müller (15), to the experiments of Aleksandrov and Lazurkin (2), which provide only the absolute value of the complex modulus as the end result of their experiments. In this investigation the complex modulus of elasticity was obtained by a mathematical transformation based on a creep-strain test, following the method presented by Papazian (18).

The basic test used was a simple flexure test under static load. It was performed on a small beam specimen with static loading applied for a specific time interval. Results from tests were evaluated in terms of the rheological characteristics of the material, following the concepts derived from the complex modulus presented by Papazian (18).

## Properties of Materials and Mix Design

The two basic soils in this investigation were designated as I-1 and K-1. The properties and gradation of these soils are given in Table 1. In both cases, an 85-100 penetration asphalt cement with a specific gravity of 0.9983 was used as the binder in the preparation of the mixtures.

In general, the Marshal method of mix design as described in ASTM D 1559-62 T was used to determine the optimum asphalt content. The only deviation from standard

procedures was that the specimens were tested at 70 F instead of the prescribed 140 F. This deviation is justified on the basis that, while 140 F is a reasonable estimate of what one might expect at the surface of a pavement, the base materials being considered in this investigation would never reach that temperature. This lower temperature is consistent with the recommendations of Kallas (11), who recommends that the testing temperature for Marshall specimens be lowered to 110 F or 100 F for asphaltic concrete mixes to be used in thicker pavement slabs. This suggests that the testing temperatures for a mix to be used strictly as a base or subbase material should be substantially lower.

TABLE 1

PROPERTIES OF SOILS INVESTIGATED

| Designation | I-1 | K-1 |
|---|---|---|
| Particle size distribution (percent passing) | | |
| ¾ in. | 100 | 100 |
| No. 4 | 74 | 86.5 |
| No. 10 | 54 | 75.5 |
| No. 40 | 22 | 20.5 |
| No. 200 | 25 | 2 |
| Liquid limit | 18.9 | |
| Plasticity index | NP | NP |

## Testing Procedure

The test specimens were 3- by 3- by 12-in. beams. The material was mixed in a heavy-duty kitchen-type mixer capable of mixing approximately 50 lb at a time. The mixer was modified to suit the needs of this investigation by placing a smaller mixing bowl fitted with a steel collar into a larger bowl. A uniform space of approximately 2 in. remaining between the bowls was filled with oil, and a gas heater was used to heat the oil. Once the heating process started, the temperature of the oil rose in a uniform manner, thus creating a uniformly heated bath for the mixing bowl itself.

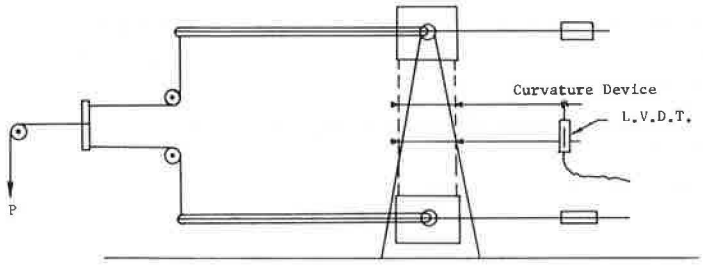
After thorough mixing of the heated constituents, the desired amount of mixture was weighed and placed in the mold. This amount was calculated from the design characteristics of the mix and the known volume of the mold so as to achieve a specified compacted density. The material in the mold was compacted under static pressure by applying compactive pressure at the top and the bottom of the mold. After compaction the mold was carefully disassembled and the compacted beam transferred onto a flat wooden board where it was allowed to cool at room temperature. Precautions were taken to avoid imposing any external stresses upon the beam specimen during cooling.

All tests performed in the course of this investigation were carried out at a specified constant temperature. This was achieved by using a constant-temperature room in which the temperature was accurately controlled within ± 0.5 F. Temperatures selected for this investigation were 40, 70, and 100 F. In order to have a uniform temperature throughout the specimen during the test, test specimens were placed in the constant-temperature room for a minimum of 24 hours before the actual testing time.

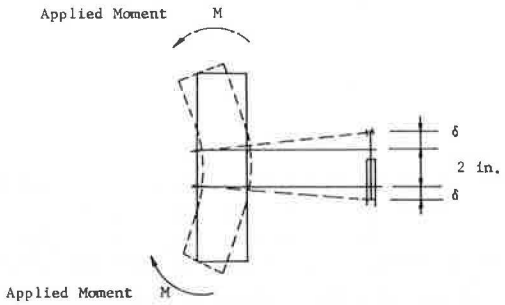
Since no standard procedure or apparatus existed for the type of test conducted, it was necessary to design a testing apparatus. This was done with the following objectives in mind: (a) the design should be mechanically simple; (b) it should be flexible enough to allow for future desirable modifications or adaptations for different load application systems (for example, dynamic loading); and (c) the testing apparatus should produce simple bending, eliminating as many complicating stress factors as possible. The flexural testing device (Fig. 1) consists essentially of a frame supporting two hinged end-boxes designed to apply equal moments to both ends of the test specimen. Equal end-moments were achieved by using a mechanical system of pulleys and levers.

The load (P) was applied manually, exercising great care to hold dynamic effects to a minimum. This method of loading, although presenting some problems, worked satisfactorily throughout the test program. Alternate methods of load application considered in the early stages of the investigation all resulted in considerable mechanical complication of the testing device, with a consequent economic penalty to the program. The consistently good results obtained by the manual method did not seem to justify more complicated designs needed for a different method of load applications.

The nature of both the data and the material being evaluated made it necessary to design special instrumentation to obtain the desired data. The apparatus designed for this purpose, called the curvature device, is also shown in Figure 1. This device




(a) Illustration of the Flexural Test




(b) Working Principle of the Curvature Device

Figure 1. Flexural testing device and the curvature device.

consisted essentially of two rigid aluminum frames fastened to the beam a fixed distance apart. A LVDT attached to the ends of the aluminum frame was calibrated to indicate the average curvature in the specimen between the aluminum frames. Output from the LVDT was recorded continuously on a strip recorder to give a continuous record of curvature with time under a constant moment.

## Sample Size and Repeatability of Results

The total number of tests carried out during this program exceeded 900. For every condition, i.e., for a given mix, load condition, and temperature, from two to four samples were tested. In some isolated cases up to six samples were tested for one particular set of conditions. In general, good agreement was obtained between repeated tests under identical conditions. In those cases where the result from subsequent test specimens differed by more than 20 percent from those of the first specimens tested, the size of the sample was increased to insure representative results.

## ANALYSIS AND PRESENTATION OF RESULTS

The concept of complex modulus of elasticity has been used widely by investigators in all branches of rheology (8) and has been an accepted standard for comparing results of rheological studies obtained by different techniques and on different materials (9). In this investigation a non-dynamic method of determining the complex modulus is used.

Papazian (17) showed that the complex modulus E* can be obtained as a function of frequency from static creep tests and an appropriate Fourier transform. The advantages of this approach are easily recognized; it eliminates the inherent complication of a dynamic test and, at the same time, provides the means of obtaining the desired results. Once E* is expressed in terms of frequency, it is merely a matter of algebraic operations to obtain the absolute value of E*, $|E^*|$, for any chosen frequency (7). This eliminates the need for testing many specimens at different frequencies.

The complex modulus of elasticity may be calculated by obtaining the response of the material to an applied stress that is constant in time, applying a Fourier transform to both stress and strain in order to express them in terms of frequency, and then taking the ratio of the transformed stress and strain functions. To achieve better accuracy, the Fourier transform of the differentiated function $\dot{\epsilon}(t)$ was used in these calculations rather than the transform of the original function $\epsilon(t)$. The function $\dot{\epsilon}(t)$ was differentiated graphically, yielding equations of the form $A_i e^{-k_i t}$ for a series of straight-line approximations of the $\epsilon(t)$ curves.

Once this process of graphical differentiation was completed, an approximation for $\dot{\epsilon}(t)$ was obtained by adding all the exponential expressions representing the straight-line approximations of the $\epsilon(t)$ curve to the constant or steady-state rate of strain $\dot{\epsilon}_{ss}$. From these expressions the rate of strain ($\dot{\epsilon}$) is expressed as function of time by

$$\dot{\epsilon}(t) = A_1 e^{-k_1 t} + A_2 e^{-k_2 t} + \ldots + A_n e^{-k_n t} + \dot{\epsilon}_{ss} \qquad (1)$$

To obtain the rate of strain in terms of frequency, a Fourier transform was applied to yield

$$\dot{\epsilon}(i) = \frac{A_1}{k_1 + i\omega} + \frac{A_2}{k_2 + i\omega} + \ldots + \frac{A_n}{k_n + i\omega} + \frac{\dot{\epsilon}_{ss}}{i\omega} \qquad (2)$$

The transform of the strain was obtained through the relation

$$\epsilon^*(i\omega) = \frac{1}{i\omega}\left[\dot{\epsilon}^*(i\omega) + \epsilon(0)\right] \qquad (3)$$

where $\epsilon(0)$ is the initial strain at $t = 0$.

Results from the testing program, however, showed little or no initial strain. Hence the $\epsilon(0)$ term approached zero. This is clearly illustrated by the typical strain-time curve shown in Figure 2. Absence of an initial strain $\epsilon(0)$ was consistent throughout this study; hence, it must be concluded that for these materials strain is a continuous

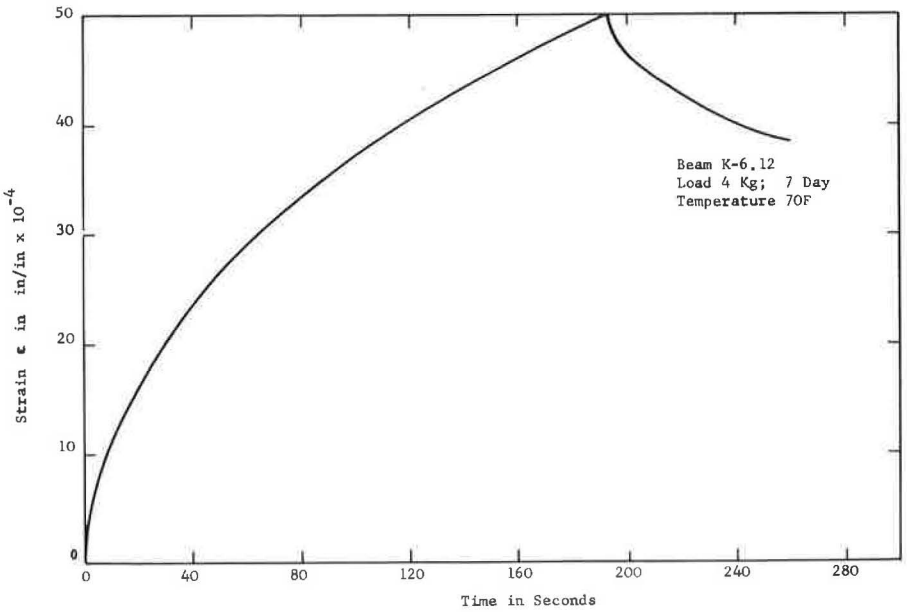



Figure 2. Typical strain-time curve for a soil-asphalt mix.

function of time, with no discontinuities at the origin and with a single point of contact at the origin. Equation 3 then becomes

$$\epsilon^*(i\omega) = \frac{1}{i\omega}\left[\dot{\epsilon}^*(i\omega)\right] \tag{4}$$

Substituting the transform of the strain rate by its equivalent expression in Eq. 4, the strain transform is given as a function of frequency by

$$\epsilon^*(i\omega) = \frac{1}{i\omega}\left[\frac{A_1}{k_1 + i\omega} + \frac{A_2}{k_2 + i\omega} + \cdots + \frac{A_n}{k_n + i\omega} + \frac{\dot{\epsilon}_{ss}}{i\omega}\right] \tag{5}$$

The Fourier transform of $\sigma(t)$ is given (22) by

$$\sigma^*(i\omega) = \int_{-\infty}^{\infty} \sigma(t) e^{-i\omega t}\, dt \tag{6}$$

The applied stress, as used in this investigation both in flexure and in tension, is equal to

$$\sigma(t) = \sigma_0 \quad \text{for } t > 0$$
$$\sigma(t) = 0 \quad \text{for } t < 0$$

where $\sigma_0$ is the applied stress due to static load.

Substituting these values in Eq. 6, it is found that the transform of $\sigma(t)$ is equal to

$$\sigma^*(i\omega) = \int_{0}^{\infty} \sigma_0 e^{i\omega t}\, dt \tag{7}$$

Integrating the expression in Eq. 7, the value of $\sigma^*(i\omega)$ becomes

$$\sigma^*(i\omega) = \frac{\sigma_0}{i\omega} \tag{8}$$

Using the basic relationship for the modulus of elasticity, the complex modulus can be expressed as

$$E^*(i\omega) = \frac{\sigma^*(i\omega)}{\epsilon^*(i\omega)}$$

Substituting the expressions for $\epsilon^*(i\omega)$ and $\sigma^*(i\omega)$ as given by Eqs. 5 and 8, the complex modulus $E^*$ is obtained as a function of frequency in the form

$$E^*(i\omega) = \frac{\sigma_0}{\frac{A_1}{k_1 + i\omega} + \frac{A_2}{k_2 + i\omega} + \cdots + \frac{A_n}{k_n + i\omega} + \frac{\dot{\epsilon}_{ss}}{i\omega}} \tag{9}$$

Equation 9 was the basic expression used to calculate the complex modulus $E^*$ from the data obtained in the laboratory testing program.

Complex Modulus as a Function of Frequency

The only difference between the expression shown in Eq. 9 and the analogous expression derived by Papazian (17, 18) is the additional initial strain value $\epsilon(0)$ that appears in the denominator of Papazian's work and as shown in Eq. 10:

$$E^*(j\omega) = \frac{\sigma_0}{\frac{A_1}{k_1 + j\omega} + \frac{A_2}{k_2 + j\omega} + \cdots + \frac{A_n}{k_n + j\omega} + \frac{\dot{\epsilon}_{ss}}{j\omega} + \epsilon(0)} \tag{10}$$

In order to express E* in terms of frequency on the real plane and to evaluate the influence that some of the variables have on this parameter, it is necessary to compute the absolute value of E*. Comparison of the absolute value of E* as determined from Eqs. 9 and 10 when plotted against frequency shows there is a fundamental difference in the two curves. Since the only difference between Eqs. 9 and 10 is the initial strain term $\epsilon(0)$ that appears only in Eq. 10, the difference must be due to this factor. To evaluate the influence of initial strain $\epsilon(0)$ on the final expression for $|E^*|$, consider the completely general expression for E*, which can be written as

$$E^* = \frac{K_0}{\sum_{k=1}^{n} \frac{A_k}{K_k + i\omega}} \tag{11}$$

Equation 11 corresponds to the expression for E* as indicated in Eq. 9. The values $K_0$, $A_1$, and $A_k$ are constants. In order to obtain the absolute value of E*, certain algebraic operations must be performed:

$$E^* = \frac{K_0}{\frac{K_k - i\omega}{K_k - i\omega} \sum_{k=1}^{n} \frac{A_k}{K_k + i\omega}} \tag{12}$$

Multiplying and dividing the denominator by the conjugate of the denominators appearing within the sum and performing the operations indicated in Eq. 12, the expression for E* can be reduced to

$$E^* = \frac{K_0 P_{2n}(\omega)}{Q_{2n-2}(\omega) - i Q_{2n-1}(\omega)} \tag{13}$$

where $P(\omega)$ and $Q(\omega)$ are polynomials in $\omega$ and the subscripts indicate the degree of the polynomials. Multiplying and dividing Eq. 13 by the conjugate of the denominator gives

$$E^* = \frac{K_0 P_{2n}(\omega) \cdot \left[Q_{2n-2}(\omega) + i Q_{2n-1}(\omega)\right]}{\left[Q_{2n-2}(\omega) - i Q_{2n-1}(\omega)\right]\left[Q_{2n-2}(\omega) + i Q_{2n-1}(\omega)\right]} \tag{14}$$

Equation 14 can be reduced to

$$E^* = \frac{K_0 P_{2n}(\omega)\left[Q_{2n-2}(\omega) + i Q_{2n-1}(\omega)\right]}{\left[Q_{2n-2}(\omega)\right]^2 + \left[Q_{2n-1}(\omega)\right]^2} \tag{15}$$

Separating the real and imaginary parts,

$$E^* = \frac{R_{4n-2}(\omega)}{M_{4n-2}(\omega)} + i\,\frac{N_{4n-1}(\omega)}{M_{4n-2}(\omega)} \tag{16}$$

where R, M, and N are again polynomials in $\omega$. The absolute value of E* is then given by

$$|E^*| = \sqrt{\left[\frac{R_{4n-2}(\omega)}{M_{4n-2}(\omega)}\right]^2 + \left[\frac{N_{4n-1}(\omega)}{M_{4n-2}(\omega)}\right]^2} \tag{17}$$

Equation 17, however, can be reduced to the form

$$|E^*| = \frac{\sqrt{L_{8n-2}(\omega)}}{M_{4n-2}(\omega)} \tag{18}$$

which in turn can be expressed as

$$|E^*| = \frac{|\omega^{4n-1}|\,|C_0|}{|\omega^{4n-2}|} \cdot \frac{\sqrt{1 + \frac{c_1}{\omega} + \frac{c_2}{\omega^2} + \cdots}}{\left|\left(1 + \frac{m_1}{\omega} + \frac{m_2}{\omega^2} + \cdots\right)\right|} \tag{19}$$

Equation 19 shows that, for $\omega$ exceeding a certain arbitrary minimum value, $|E^*|$ can be approximated by

$$|E^*| = |C_0| \cdot |\omega| \tag{20}$$

This approximation represents a linear relationship between $|E^*|$ and $|\omega|$ passes through the origin. This approximation is valid for values of $\omega$ as small as 0.5 radians per second.

If a similar procedure is followed starting with the expression of E* as given by Papazian, Eq. 10, it can be shown analytically that the resulting value of $|E^*|$ can be approximated by a constant for values of $\omega > N$, where N is an arbitrarily positive number.

In that case, the absolute value of E* can be expressed as

$$|E^*| = \frac{|c_0| \cdot |\omega^{4n}|}{|\omega^{4n}|} \cdot \frac{\sqrt{1 + \frac{c_1}{\omega} + \frac{c_2}{\omega^2} + \cdots}}{\left|1 + \frac{k_1}{\omega} + \frac{k_2}{\omega^2} + \cdots\right|} \tag{21}$$

which will lead to an approximation of $|E^*|$ given as a constant value

$$|E^*| = |C_0| \tag{22}$$

for all $\omega > N$.

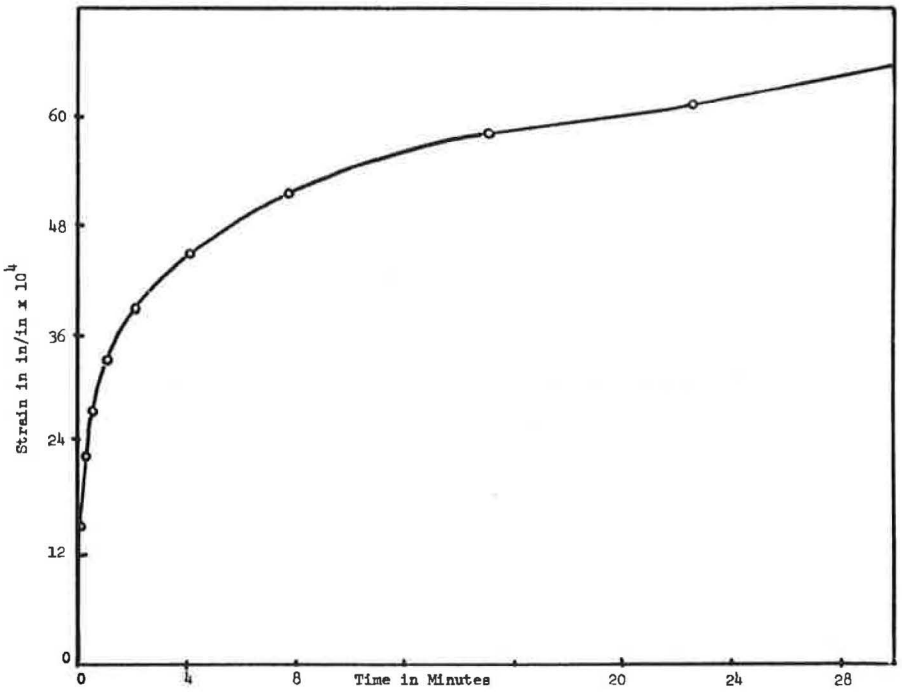



Figure 3. Typical strain-time curve as obtained by Papazian.

Comparing Eqs. 20 and 22, it can be seen that the absolute value of the complex modulus as given by these expressions is radically different. Eq. 9 for $\epsilon(0) \rightarrow 0$ indicates that $E^*$ is a linear function of frequency ($\omega$) for all values of $\omega$ above a specified minimum value. Equation 10, for real values of $\epsilon(0)$, indicates that $E^*$ approaches a constant value independent of the frequency ($\omega$) above a specified maximum value of $\omega$. These findings demonstrate the critical nature of the initial portion of the creep-strain curve on the final results when determining the complex modulus. Items that appear minor when evaluating the creep-strain data—for example, the scale of the time axis of the time strain plot—can have a significant effect on the final answer obtained by this procedure.

## RESULTS OF INVESTIGATION

### Rheological Behavior of Bituminous Mixtures

Although the majority of the findings of this investigation are the result of experimental work, a significant contribution toward understanding the behavior of the materials under study was obtained through a theoretical analysis such as that presented in the preceding section.

This investigation indicates that the response of the bituminous mixture to applied stress is highly viscous in nature. This conclusion is based on the static tests performed during this investigation and the dynamic behavior of the material as predicted by the procedures described.

As was pointed out, the presence or absence of an initial strain in the strain-time curve of the material plays an important role in the determination of the predicted behavior of the material under dynamic load. Figures 2 and 3 show typical strain-time curves as obtained in this study and by Papazian. It can be seen that the curve obtained in this investigation shows no initial strain; however, the curve obtained by Papazian (Fig. 3) exhibits a large initial strain at $t = 0$. It must be observed, however, that the initial strains shown in Figure 3 may be due more to the condensed time scale than to a true initial strain. Figure 2, which shows no initial strain, is plotted over a total time interval of approximately 3 minutes, whereas in Figure 3 this same distance on

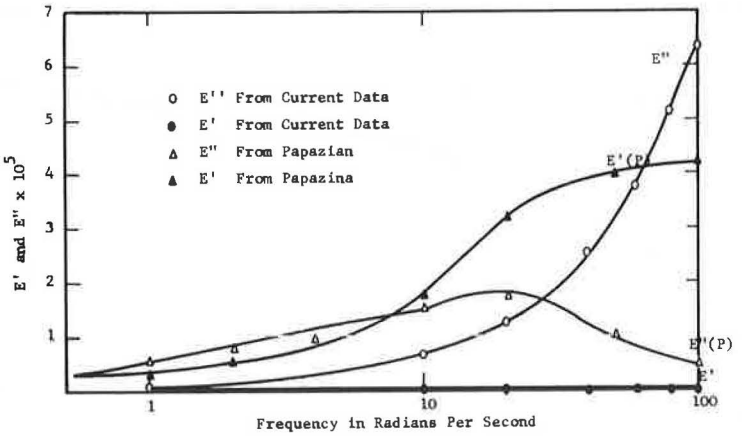



Figure 4. Comparison of real and imaginary parts as given by this study and by Papazian.

the time scale represents nearly 18 minutes. If the time scale in Figure 2 were condensed by a factor of 6 the curve would surely show an apparent $\epsilon(0)$ due simply to the inability to show small strains on the scale used. Thus the two curves in Figures 2 and 3 are more nearly identical than would be indicated by the curves shown.

Presence of an initial strain $\epsilon(0)$ (Fig. 3) indicates an immediate and elastic response of the material to an applied load, but even more important is the role that

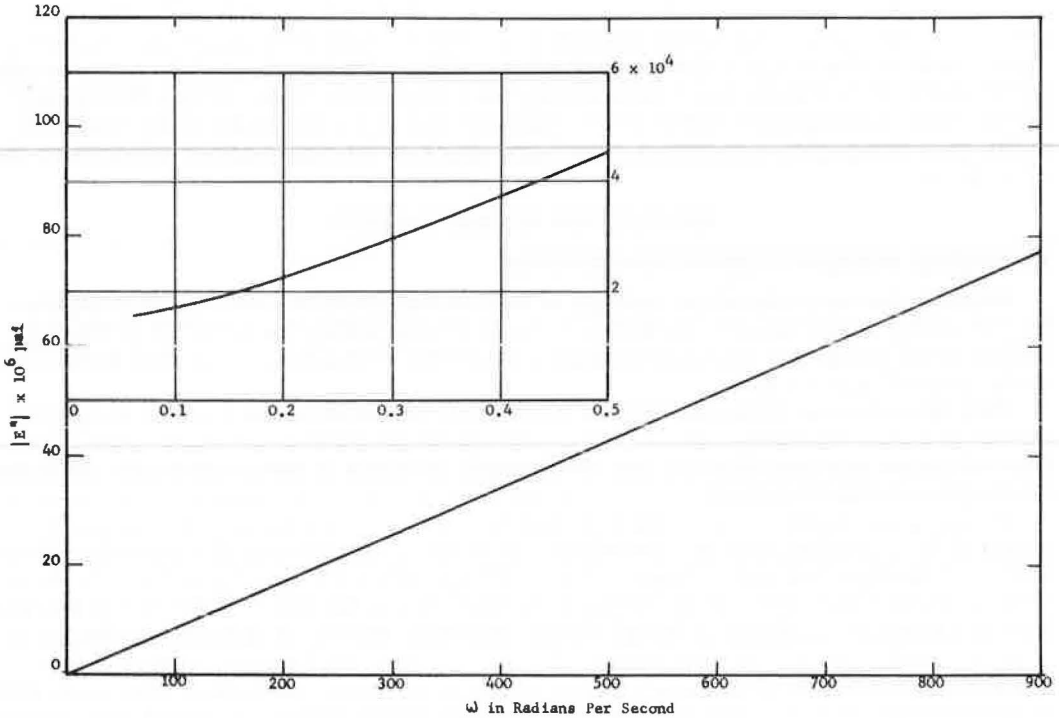



Figure 5. The complex modulus $|E^*|$ as a function of $\omega$.

initial strain plays in determining the relationship between $|E^*|$ and the frequency $\omega$. By virtue of the position $\epsilon(0)$ in Eq. 10, when the necessary algebraic operations are performed to compute the complex modulus $E^*$ or its absolute value $|E^*|$, the presence of $\epsilon(0)$ in that equation makes the real component of $E^*$ the dominant of the two components in the complex expression of $E^*$. On the other hand, when the initial strain is small, as was found to be the case in this study, the real part $E'$ of the complex modulus $E^*$ remains practically constant throughout the range of frequencies, while its imaginary counterpart $E''$ assumes the dominant role and increases very rapidly with an increase in frequency.

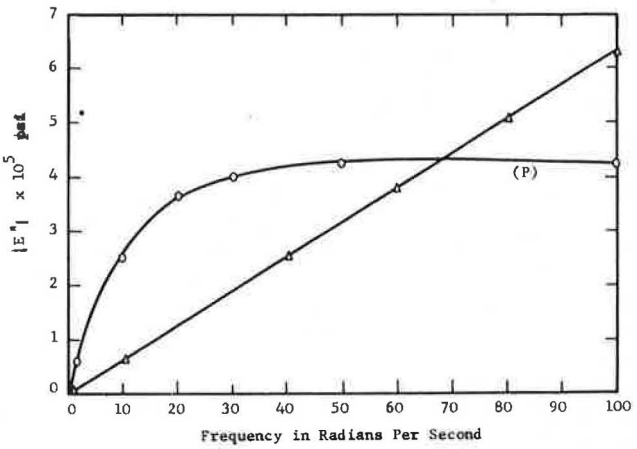



Figure 6. Comparison of $|E^*|$ as a function of frequency.

The important role played by the inital strain value $\epsilon(0)$ in determining the nature of the response of the material under study is shown in Figure 4. In Papazian's results

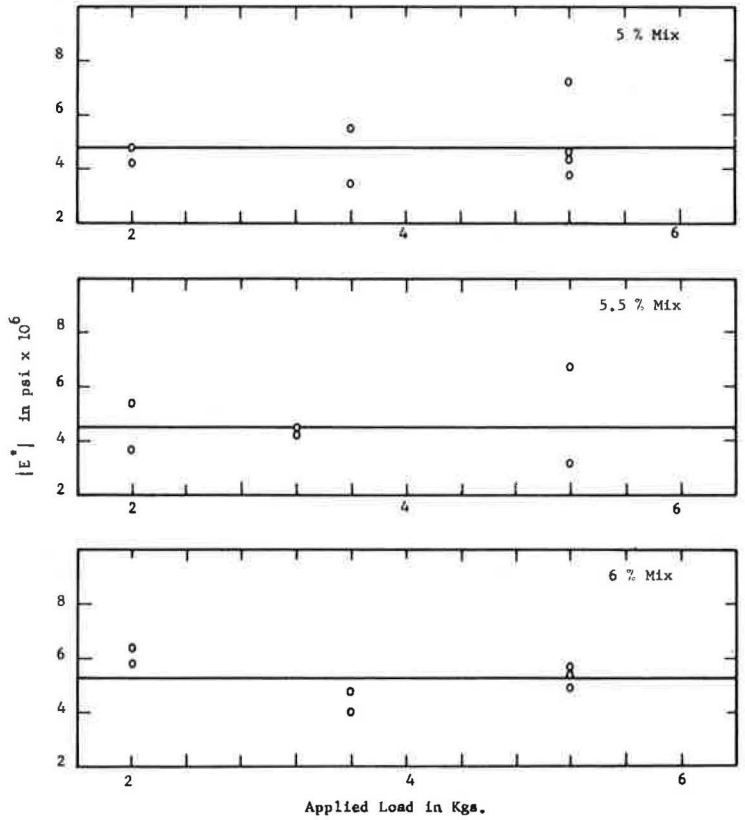



Figure 7. Effect of the applied load (I-I Soil, 70 F).

it can be seen that the real component of the complex modulus is the more important of the two components. The results of this study show, however, that the real component of the complex modulus E* is very small and remains constant throughout the range of frequency values, while the imaginary component E* increases continuously with frequency increase.

The significance of this becomes even more apparent when it is considered that the response of an elastic material is completely in phase with the applied stress, while the response of a viscous material is 90 deg out of phase with the stress (6). The real part of the complex modulus E* is, in effect, a measure of the elasticity of the material, while the imaginary part is a measure of its viscosity. It can be concluded, then, from the results of this study that bituminous mixes such as studied here have a very low elastic response and a high viscous response to applied stress. Furthermore, if the total response of the material were measured by the absolute value |E*| of its complex modulus, it would be found that |E*| will be in an approximately linear relationship with the frequency of applied load (Fig. 5). Since these values of |E*| are predicated values based on a theoretical approach with no upper limit applied, some of the values are exceedingly high. The higher values that exceed the conceivable values of the aggregate itself are due to the high projected frequencies used in the analysis.

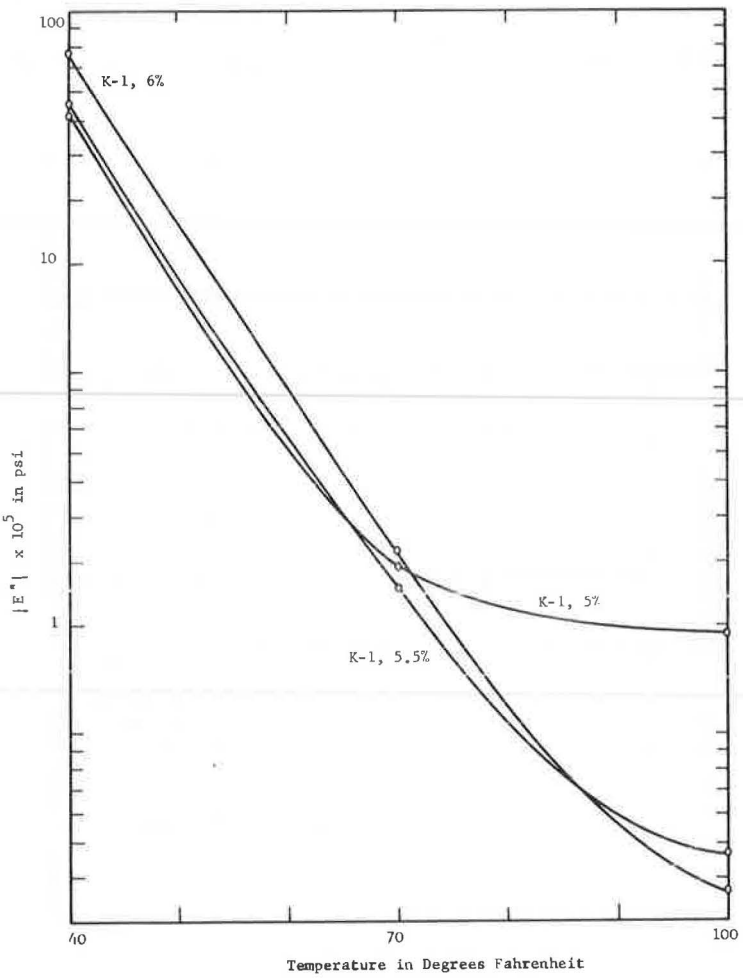



Figure 8. Effect of temperature.

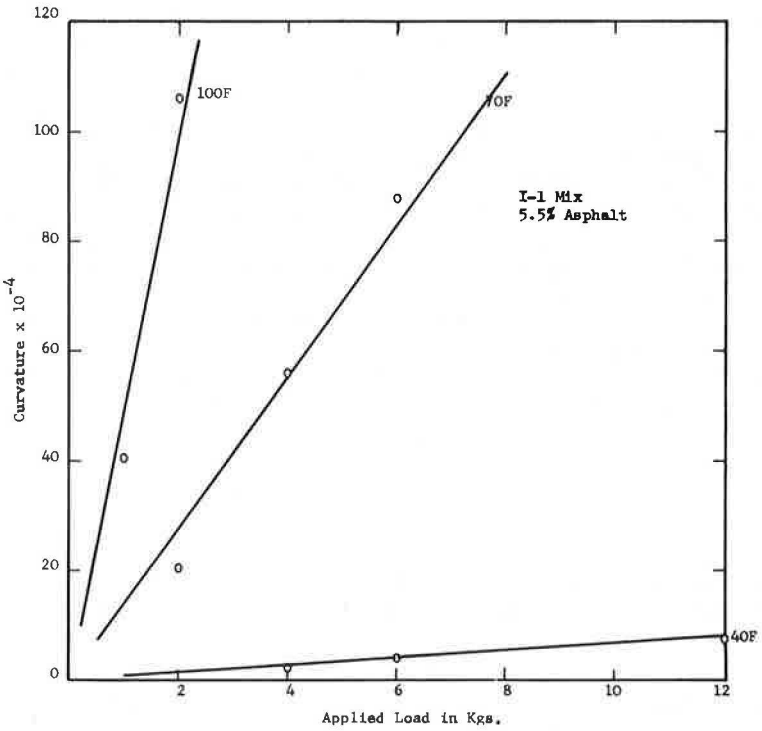



Figure 9. Effects on curvature of applied load and temperature.

These higher frequencies are well beyond values that can be expected under normal loading conditions in a pavement system. A comparison between $|E^*|$ and frequency for the current study and the results reported by Papazian (17) are shown in Figure 6. The difference between these results is due to the influence exerted by the initial strain value $\epsilon(0)$.

Effect of Applied Stress on $|E^*|$

The results from this investigation show that the complex modulus $|E^*|$ calculated for a given frequency is independent of the applied stress. These results are shown in Figure 7 for three different mixtures. Although scattering of data exists, statistical analysis performed on experimental data shows no statistically significant differences in these results. Analogous results were obtained for other mixtures tested.

Effects of Temperature and Binder Content in the Mix on $|E^*|$

The influence of temperature and the amount of binder in the mix on the rheological behavior of bituminous mixes is presented in terms of their effects on the complex modulus $|E^*|$. Representation of the influence exerted by these factors is shown in Figures 7 through 9.

The general trend for both of these factors is seen as a decrease in $|E^*|$ with an increase in either temperature or binder content. The influence of the bituminous binder content is seen to increase at the higher temperatures.

## CONCLUSIONS

The literature review clearly emphasized the need to evaluate the bituminous mixtures as viscoelastic materials. Since viscoelastic materials are sensitive to rate of loading, it is necessary to evaluate these materials under variable loading conditions.

Following the lead of Papazian (17, 18), the creep-strain tests with static loading were used to develop data for predicting the response of the bituminous materials under dynamic loading. Tests were conducted to evaluate techniques used in collecting and interpreting data collected from the creep-strain tests.

Findings from this study show the need for careful interpretation of the creep-strain data to produce meaningful data for design. The characteristic shape of the time-strain curve immediately after loading has a critical influence when interpreting the data. Knowledge of the response of the material immediately after loading is highly critical because of its influence on the projected influence on the behavior of the material under variable dynamic loading conditions and because of the transient type loading that occurs in most pavements in service.

Additional data are needed to evaluate more critically the short-duration loading of bituminous materials and the use of creep-strain tests to evaluate these materials. If, as indicated in this study, the bituminous materials are more nearly viscous than viscoelastic, then efforts must be made to determine a limiting value for the complex modulus. Absolute values for the complex modulus greater than the modulus of elasticity of the aggregate portion of the mixture are obviously inaccurate. Thus, an upper limiting value must be determined for the complex modulus of bituminous mixtures.

## ACKNOWLEDGMENTS

This study was conducted as a part of the National Cooperative Highway Research Program, Project 4-1. Additional reports from this project are being prepared.

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# Rheological Aspects of Aging: Part II

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The effect of aging of the asphaltic binder on the rheological response of sand-asphalt mixtures is studied. The experiment and the method of analysis follow procedures outlined in an earlier report (4). The creep response of sand-asphalts at various temperatures and aging have been analyzed, and the viscoelastic model parameter and overall response have been related to the asphalt aging index. Differences are shown in the aging susceptibility of mixtures prepared with different asphalts. Expressions are presented to show the effect of asphalt aging index on the viscoelastic model parameter at various test temperatures. Analysis of activation energy has been carried out that show that there are certain limitations in using the activation energy concept in the study of aging.

•BITUMINOUS mixtures are composite systems of a heterogeneous granular matrix and a viscoelastic binder designed to perform three major specific functions: provide structural stability under traffic load, withstand environmental effects and thus retain the desired structural stability, and withstand the effect of repeated wheel load applications during the service life of the pavement. Besides these structural functions, there are also other requirements based on traffic safety considerations.

In a rational approach to pavement design, the pavement is considered as a system incorporating the three structural functions to predict pavement performance at a designated service life. Developing this method of pavement analysis requires three separate design criteria: stability, durability, and resistance to fatigue. These criteria are then incorporated into a unique three-dimensional relation expressed in terms of material properties.

The ultimate objective in durability design is to develop relations expressing changes in the physical or rheological properties of mixtures as they relate to the basic properties of the mix constituents. All bituminous mixtures change with time in one way or another, depending on the loading and environmental conditions. These changes are primarily associated with the aging susceptibility of the binder and its interaction with the mineral aggregate.

The aging characteristics of the bituminous binders have been discussed in great length in previous reports (1, 2, 3, 4). It is generally agreed that exposure of bituminous materials to the service environment results in changes in the properties of the binder. Laboratory and field investigations indicate that the asphaltic binders harden during service life (5, 6, 7).

It has also been reported that asphalt hardening is associated with changes occurring in chemical constituents of the binder. It is generally agreed that binder aging results in an increase in the asphaltene content at the expense of the other components (8, 9, 10).

With respect to the changes in the rheological properties, it has been shown that the viscosity of the binder is also affected by the exposure to the aging environment (1). Viscosity may be represented by an Arrhenius form equation,

Paper sponsored by Committee on Characteristics of Bituminous Materials.

$$\eta = \frac{hN}{V} e^{\frac{-\Delta S}{R}} e^{\frac{\Delta E}{RT}}$$

where

$\eta$ = coefficient of viscosity,
h = Planck's constant,
V = molar volume,
N = Avagadro's number,
$\Delta S$ = entropy, and
$\Delta E$ = activation energy.

Therefore, the change in viscosity due to aging is a result of the change in activation energy, entropy, or the molar volume. Since the molecular weight of asphalt does not change significantly with aging, then it has been argued that the aging is related to the change in activation energy and entropy. Expressing the change in entropy in terms of heat, $\Delta q$, and aging temperature, $T_a$, it is shown that

$$\Delta S_a - \Delta S_u = d\Delta S = \frac{\Delta q}{T_a}$$

or

$$\eta_a = \frac{hN}{V} \exp\left(\frac{-\Delta S_u}{R}\right) \exp\left(\frac{-\Delta q}{RT_a}\right) \exp\frac{\Delta E}{RT}$$

where indices a and u refer to aged and unaged conditions.

The asphalt aging index, which is the ratio of aged to unaged viscosity, can be expressed by

$$AI = \frac{\eta_a}{\eta_u} = \exp\left[\frac{(\Delta E_a - T\Delta S_a)}{RT} - \frac{(\Delta E_u - T\Delta S_u)}{RT}\right]$$

or

$$AI = \exp\left(\frac{\Delta F_a - \Delta F_u}{RT}\right)$$

This equation indicates that the aging characteristics of the bituminous binder can be expressed in terms of changes in free energy.

Other rheological parameters are also affected by aging. The investigation by Sisko (8) indicates that the complex shear modulus G* ($\omega$) and elastic and viscous dynamic moduli are affected by the aging process. Sisko also reported that the ratio of complex modulus G* ($\omega$) for aged and unaged conditions can be correlated with the field performance of pavements. It is reported that the high ratios of $G^*_a/G^*_u$ are associated with the cracking in the field, whereas the low ratios are related to roads exhibiting plastic deformations.

With respect to the aging of bituminous mixtures, it was reported previously that the creep response of sand asphalt mixtures exhibit age dependency (3, 4). It has been shown that the temperature dependence function, $a_T$, of the mixture similar to the asphalt binder is affected by the aging condition. There is also an apparent correlation between the aging susceptibility of the binder and the aging susceptibility of the mixtures measured by the mixture viscosity. In this report these observations are further substantiated by additional experimental data, and the age dependency of viscoelastic parameters and other pertinent parameters are discussed.

## MATERIALS AND PROCEDURES

Four different types of asphalt cements were used in this investigation. They are the same materials that were used by Moavenzadeh and Stander (1) and Lottman and Rao (2) in a study of durability characteristics of asphaltic materials. These asphalts are designated as 60/70 penetration grade from a Venezuelan crude, two AC-20 as-

TABLE 1

TEST CHARACTERISTICS OF ASPHALT CEMENTS

| Test | 60/70 | B-3056 | AC-20 | B-2960 |
|---|---|---|---|---|
| Specific gravity | 1.010 | 1.020 | | 1.034 |
| Softening point, ring and ball | 123 F | | 127 F | 125 F |
| Ductility, 77 F | 150°+cm | 150°+cm | 150°+cm | |
| Penetration: 100 gm, 5 sec, 77 F | 63 | | | |
| 200 gm, 60 sec, 39.4 F | 23.5 | 30 | | |
| Flash point, Cleveland open cup | 455 F | 545 F | | 515 F |

phalts coded B-2960 and B-3056 obtained from the Asphalt Institute and Bureau of Public Roads cooperative study of viscosity graded asphalts, and a third AC-20 asphalt cement. Test characteristics of these asphalts are given in Table 1.

The aggregate used in this study was an Ottawa sand with a powdered silica filler. To prepare the aggregate, coarse-graded and fine-graded Ottawa sands were sieved into No. 16, 30, 50, 100, and 200 mesh, and then recombined to form the desired gradation (4).

Aging of the asphalt binder was carried out with a modified thin film oven procedure developed by Vallerga (11) and described previously (3, 4, 1). For aging, the dishes containing the asphalt binder were placed on the shelves of the aging oven, which can maintain the desired temperature with ± 5 F accuracy. The following table gives the aging temperature and the duration of aging used.

| Test Temperature (F) | Duration (hours) |
|---|---|
| 325 | 3, 9 |
| 375 | 3, 9 |
| 425 | 7 |

The sample preparation followed the procedures outlined earlier (4). Specimens were prepared using Harvard miniature molds of 1.312 by 2.624 in. The sand-asphalt mixtures were compacted using a double plunger arrangement and a static load of 3,000 psi that was maintained for 2 min. A compression apparatus used for the Marshall test method was modified to provide the required compactive effort.

These specimens were then subjected to creep tests (3, 4). The creep experiments were conducted at 15, 25, 45, and 60 C, using stresses of 20, 10, 5, and 2 psi throughout the investigation, except for the AC-20 asphalt at 45 C where stress was reduced to 20.0 psi.

## ANALYSIS AND DISCUSSION OF RESULTS

### Asphalt Aging Index

The primary objective of this investigation is to relate the durability characteristics of the asphaltic binders to the rheologic responses of the bituminous mixtures. To achieve such an objective, it is necessary that the age susceptibility of the binder used in the sand-asphalt mixture be determined as accurately as possible. In the investigation conducted by Moavenzadeh and Stander (1), it was shown that the asphalt aging index measured at different temperatures and at various aging conditions could be used as a measure of age susceptibility of asphaltic materials. However, the experimental results presented by these investigators showed a relatively high degree of scatter at certain aging conditions. In order to verify the available data, the four asphalt

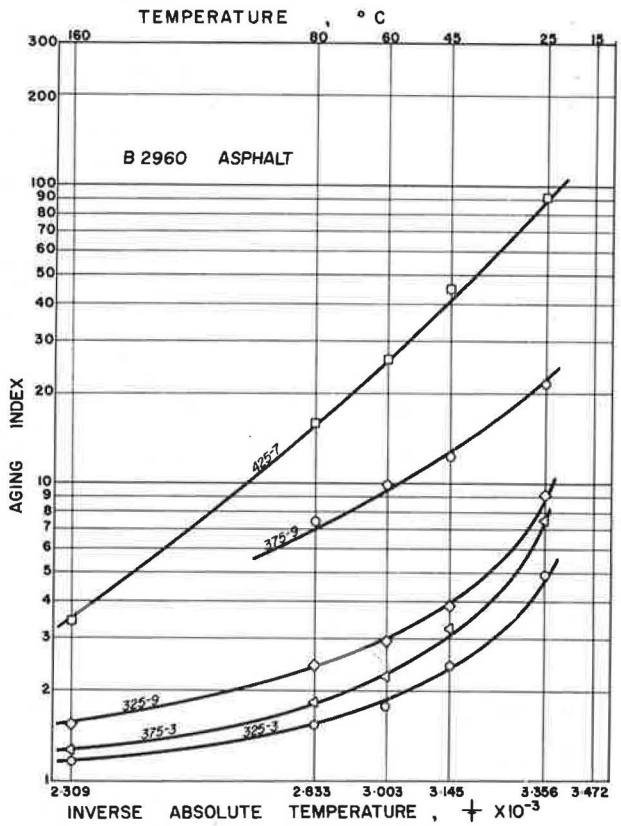


Figure 1. Relationship between asphalt aging index and temperature.

cements were aged, and the aging index at a given test temperature and aging condition was determined as

$$AI = \frac{\eta \text{ aged}}{\eta \text{ unaged}}$$

The viscosity values in this equation were determined by plotting shear stress rates of shear data on a log-log plot and using the power input method of $\tau\gamma = 1{,}000$.

In Figures 1 and 2, the variations of asphalt aging index with temperature at various agings and for two different asphalts are shown. From these figures one could observe that, first, the aging susceptibility of asphalts increases with the degree of aging used, and second, for the same aging condition, as the test temperature decreases the asphalt cements exhibit a greater degree of age susceptibility. Furthermore, the aging index and temperature are related by

$$AI = \exp \frac{\Delta F_a - \Delta F_u}{RT}$$

These figures show that the change in the free energy due to aging, $\Delta F_a - \Delta F_u$, does not remain constant at different test temperatures. Moavenzadeh and Stander, by fitting a straight line through their data, attempted to show that $\Delta F_a - \Delta F_u$ is a constant at different temperatures and is only affected by the degree of aging. The experimental data shown in Figures 1 and 2, however, indicate that, although at very high temperatures this assumption may be an approximation, at low temperatures $\Delta F_a - \Delta F_u$ is a temperature-dependent function. This phenomena is attributed to the

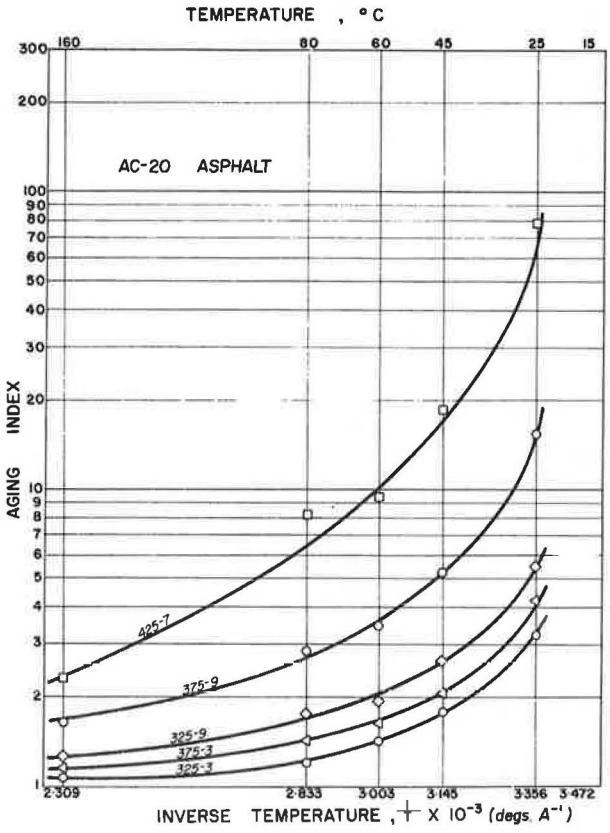


Figure 2. Relationship between asphalt aging index and temperature.

sharp increase in the activation energy at low temperatures, which is related to the decrease in the free volume of the system. It has been shown that the mobility at any temperature primarily depends on the free volume available, which in turn is related to the energy barrier required for the whole formation.

Furthermore, the comparison of aging index temperature relations of four asphalts at the constant aging conditions of 375-3, 375-9, and 425-7 indicates that asphalt B-2960 exhibits greater aging susceptibility at almost all aging conditions, whereas the aging susceptibility of the other asphalts varies to such an extent that their comparisons are impossible.

## Aging Response of Mixtures

To investigate the effect of aging on the creep response of sand-asphalt mixtures, various components of time-deformation relations have been analyzed. These include the viscoelastic parameters, creep curves, and the activation energy relations. In the following sections these analyses are discussed.

Viscoelastic Parameters—The creep response of rheological systems such as asphalt-aggregate mixtures is governed by three deformation mechanisms: instantaneous elastic, viscous flow, and retarded elastic. In viscoelastic analysis, the instantaneous deformation is considered as a reversible process that can be fully recovered upon removal of external load. However, in asphaltic mixtures or in similar systems due to specimen seating or internal aggregate interlocking mechanisms, the instantaneous deformation may also include some irreversible deformation. In this study the effect of seating conditions has been minimized by using special testing procedures (4). The second deformation mechanism is viscous flow, which is proportional to the slope of the steady-state portion of the creep curve. Many engineers, based on field observation, doubt that asphaltic mixtures reach a steady-state flow condition.

These observations appear to be valid for dense graded mixtures. However, for sand-asphalt mixtures, the experimental data indicate that within the experimental errors a constant deformation rate is reached between 300 and 400 seconds. One should exercise great caution in the use and interpretation of these data. The extrapolation of the steady-state flow condition to times greater than the experimental time span is not recommended.

The third portion of creep response is retarded elastic behavior. The observed delayed deformation can be characterized by two separate means: the delayed deformation can be represented by discrete numbers of retarded elastic mechanisms, each having a particular retardation time, or it can be assumed that an infinite number of retarded deformation mechanisms contribute to the creep process. In this case, the

retardation time is a continuous function of time and is represented by a distribution function of the retardation times.

The mathematical expression for the creep response of sand-asphalt mixtures with discrete retarded elastic deformation mechanisms is given by

$$\epsilon = \frac{\sigma_o}{E_o} + \frac{\sigma_o}{\lambda_o} t + \sigma_o \sum_{i=1}^{i=n} \frac{1}{E_i}\left(1 - e^{-t/\tau_i}\right)$$

where

$\sigma_o$ = constant stress,
$E_o$ = modulus of elastic deformation,
$\lambda_o$ = coefficient of viscous flow,
$E_i$ = elastic moduli of retarded deformation,
$\lambda_i$ = viscous coefficient of retarded mechanism,
$\tau_i$ = retardation time = $\lambda_i/E_i$, and
n = number of discrete retarded mechanisms.

In this analysis of the number of discrete retarded deformation mechanisms as well as the viscoelastic parameters, $E_i$, $\lambda_i$, $E_o$, and $\lambda_o$, have been determined directly from the creep curves. The results indicate that the viscoelastic parameters are related to the aging index of asphalt cements. These relations (Figs. 3 through 6) indicate that the viscoelastic parameters $E_o$, $\lambda_o$, $E_1$, $E_2$, $\lambda_1$, and $\lambda_2$ can be expressed in terms of aging index by

$$(E_i, \lambda_i) = C\,(AI)^n$$

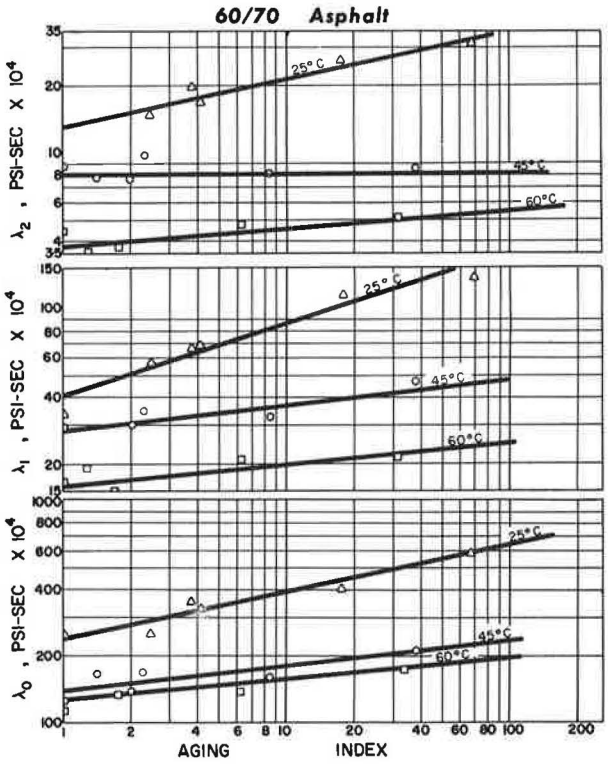


Figure 3. Effect of asphalt aging index on viscoelastic parameters.

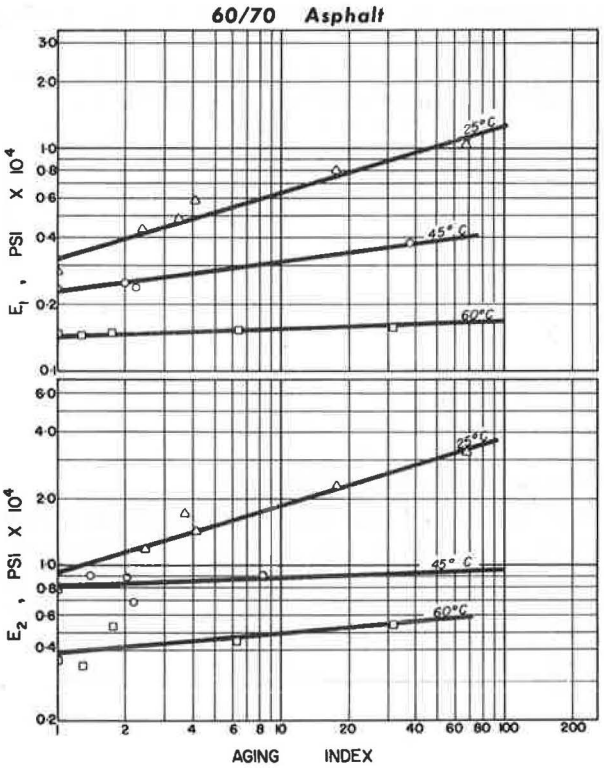


Figure 4. Effect of asphalt aging index on viscoelastic parameters.

where C is a constant corresponding to the viscoelastic parameters at unaged conditions (AI = 1.0). The constant n is considered as the viscoelastic age susceptibility constant. The results, however, indicate that the parameter $E_0$ is not significantly affected by aging.

These analyses indicate that the viscoelastic age susceptibility constant n varies by temperature, type of asphalt, and deformation mechanism involved. As shown in Figures 3 through 6, the constant n is generally increased with decreasing temperature, indicating that viscoelastic response of asphaltic mixtures becomes more dependent on the asphalt aging index at lower temperatures. At a constant temperature, the constant n varies among different viscoelastic parameters for different asphalts studied. For example, for the parameter $\lambda_0$ measured at 25 C, the susceptibility constant n attains the largest value for B-2960 asphalt (Table 2). Parameter $\lambda_0$ is determined from the steady-state portion of the creep curve and is indicative of viscous resistance of bituminous mixtures.

In recalling the variation of asphalt aging index with temperature for various asphalts shown in Figures 1 and 2, it is observed that B-2960 asphalt is also the most age-susceptible binder. This indicates, apparently, that for the same deformation mechanism it is possible to interrelate the age susceptibility of binder and that of bituminous mixture. However, when a similar comparison is made between the parameters $E_1$ and $E_2$, $\lambda_1$ and $\lambda_2$ at a given temperature (Table 3), it is observed that B-3056 exhibits greater age susceptibility n than other asphalts (larger n values). On the other hand, the AC-20 asphalt has the least age dependency with smaller n values.

In short, from the viewpoint of the effect of aging on viscous deformation, B-2960 is the most age-susceptible, whereas with respect to the retarded deformation, B-3056 exhibits the greatest effect.

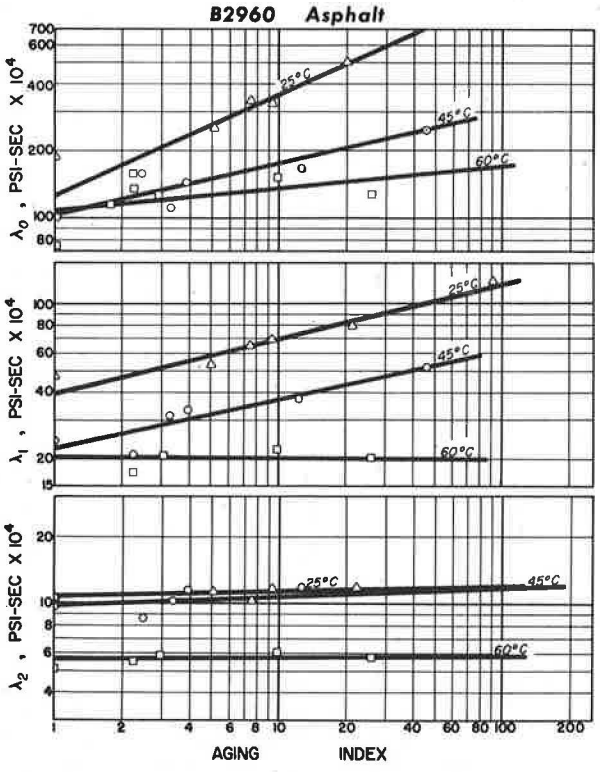


Figure 5. Effect of asphalt aging index on viscoelastic parameters.

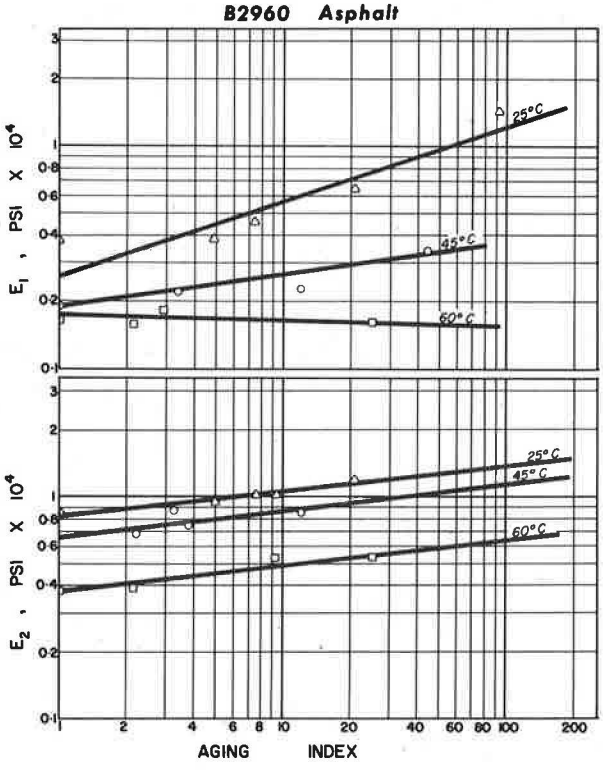


Figure 6. Effect of asphalt aging index on viscoelastic parameters.

TABLE 2

VISCOELASTIC AGE SUSCEPTIBILITY CONSTANT n FOR $\lambda_o = C\,(Al)^n$

| Asphalt | n at 25 C | n at 45 C |
|---|---|---|
| B-2960 | 0.4641 | 0.2141 |
| 60/70 | 0.2059 | 0.1192 |
| B-3056 | 0.1690 | 0.1514 |
| AC-20 | 0.02 | 0.1080 |

TABLE 3

CONSTANT n FOR RETARDED DEFORMATION PARAMETERS AT 25 C

| Asphalt | $E_1$, psi | $E_2$, psi | $\lambda_1$, psi-sec | $\lambda_2$, psi-sec |
|---|---|---|---|---|
| B-3056 | 0.4738 | 0.5035 | 0.4685 | 0.2497 |
| 60/70 | 0.2974 | 0.3074 | 0.3173 | 0.2131 |
| B-2960 | 0.3183 | 0.0981 | 0.2478 | 0.010 |
| AC-20 | 0.0892 | 0.20 | 0.0992 | 0.12 |

As pointed out, the delayed viscoelastic effect can be represented by a distribution function of retardation times. The mathematical expression for the creep response with a continuous function of retardation time is given by

$$\epsilon(t) = \frac{\sigma_o}{E_o} + \frac{\sigma_o}{\lambda_o} t + \sigma_o \int_{-\infty}^{+\infty} L\,(\ln \tau) \left(1 - e^{-t/\tau}\right) d\,(\ln \tau)$$

The distribution function L (ln $\tau$) determined by using the first-order graphical approximation technique discussed by Alfrey (12) and Ferry (11) has been used in this study to show the effect of aging on the bituminous mixtures. Figure 7 shows a typical distribution function L (ln $\tau$) for one asphalt at various aging conditions. This figure indicates the degree of aging effects on the distribution of retardation time.

Creep Response—In this section the effect of aging on the overall creep response is presented. In a previous report (4) the creep response of two asphalts at different temperatures and various aging conditions was shown. The creep response of other as-

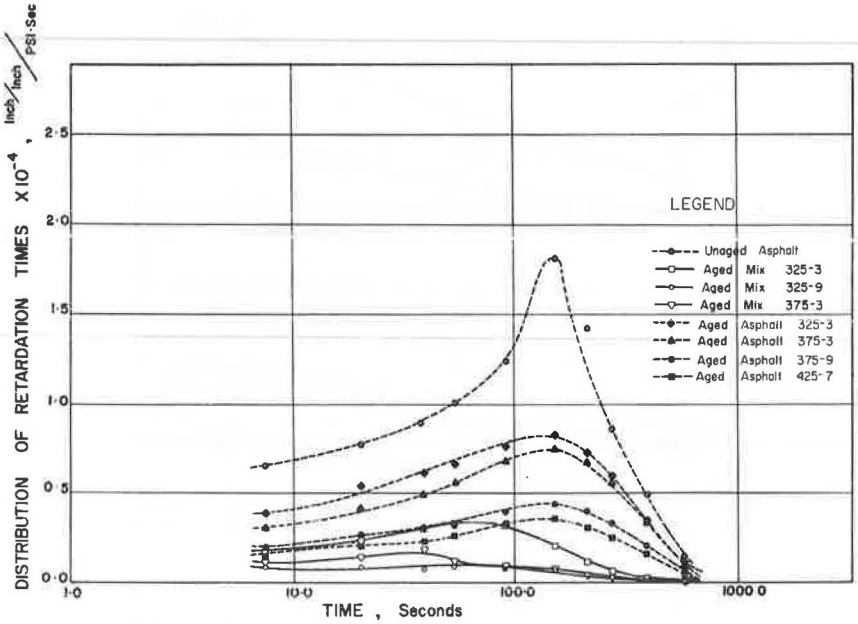


Figure 7. Distribution of retardation times for sand-asphalt (60/70).

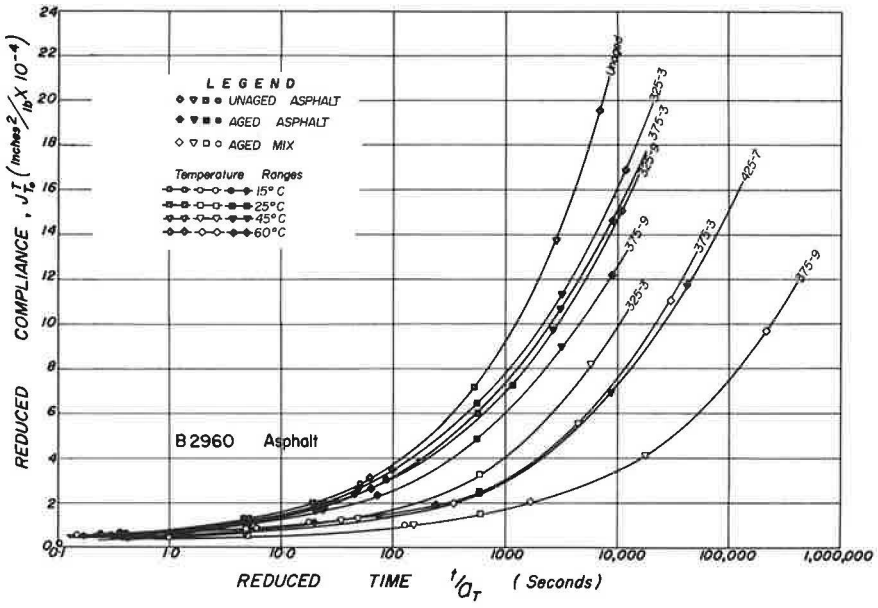


Figure 8. Master creep curve for sand-asphalt (60/70).

phalts at all temperatures and aging follows a similar pattern (Fig. 8). This indicates that, at a given temperature, the stiffness of the mixture increases with an increase in the degree of aging. That is, the compliance of mixture defined as $J(t) = \epsilon(t)/\sigma_o$ is affected by the degree of aging.

These results also indicate that the creep curves are arranged in a logical order with respect to the effect of aging. However, it should be pointed out that a statistical analysis has not been carried out to show whether there is any significant difference between the individual creep curves. In fact, it appears that certain aging conditions may be considered to have the same effect on the creep response. For example, the difference between the creep curves for 375-3 and 325-9 is not very large, and the statistical analysis may show that these differences are not statistically significant.

In the experimental characterization of viscoelastic data, the results are often presented in the form of a master curve in which data corresponding to various temperatures are reduced to a reference temperature scale. The technique of determining a master curve has been discussed in detail in the literature. It primarily consists of measuring the shift necessary to make creep curves at different temperatures coincide to the reference temperature, $T_o$. In Figures 9 and 10, the master creep curves for asphalts studied are shown. As observed, B-3056 asphalts yield a slightly stiffer mixture corresponding to a smaller compliance, J(t). In Figure 9 the master creep curves of these asphalts at 425-7 are compared. Note that the stiffness of AC-20 asphalt mixtures is not greatly changed. However, there is a definite reduction in the creep compliance of the other three asphalts, with B-3056 showing the greatest reduction. In the previous section related to viscoelastic analysis, it was pointed out that the age susceptibility constant n varies among asphalts. The results presented in Figure 10 also confirm such observations, and show that AC-20 asphalt is the least and B-3056 is the most age-susceptible asphalt.

Activation Energy—The experimental and theoretical analyses indicate that creep of material bodies is controlled by the thermally activated processes. These processes in turn are dependent on temperature and activation energy $\Delta E$, which corresponds to the energy barrier required for deformation process. Depending on the complexity of the creep phenomena and the number of micro-mechanisms involved, the activation energy may attain a wide range of values. When two micro-mechanisms operating

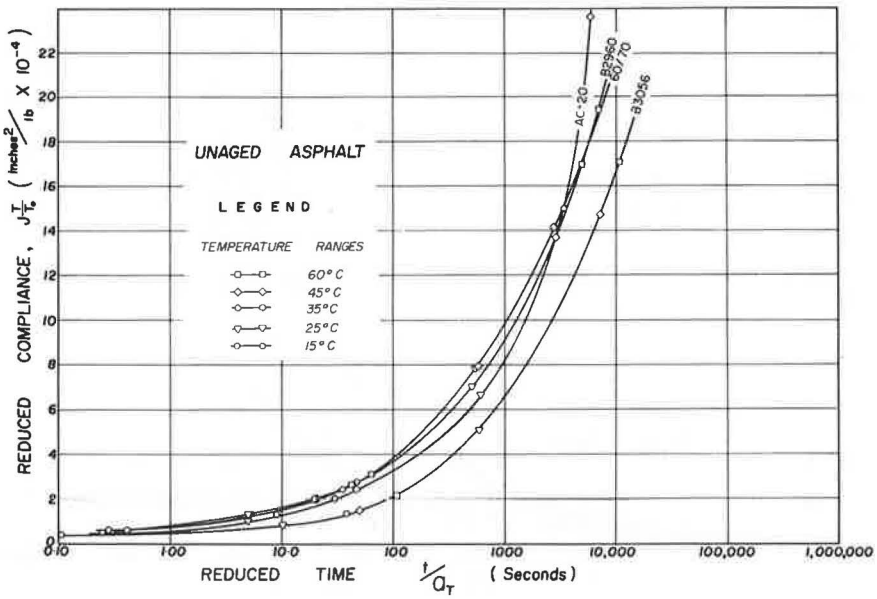


Figure 9. Master creep curves for four asphalt cements of constant aging condition.

depend on each other, the process with the largest activation energy controls the deformation. On the other hand, the reverse is true when the micro-mechanisms are independent of each other. Therefore, in a complex body it may not be possible to determine explicitly the activation energy $\Delta E_i$ for each process. Instead, an apparent activation energy can be determined to represent the deformation of the material body.

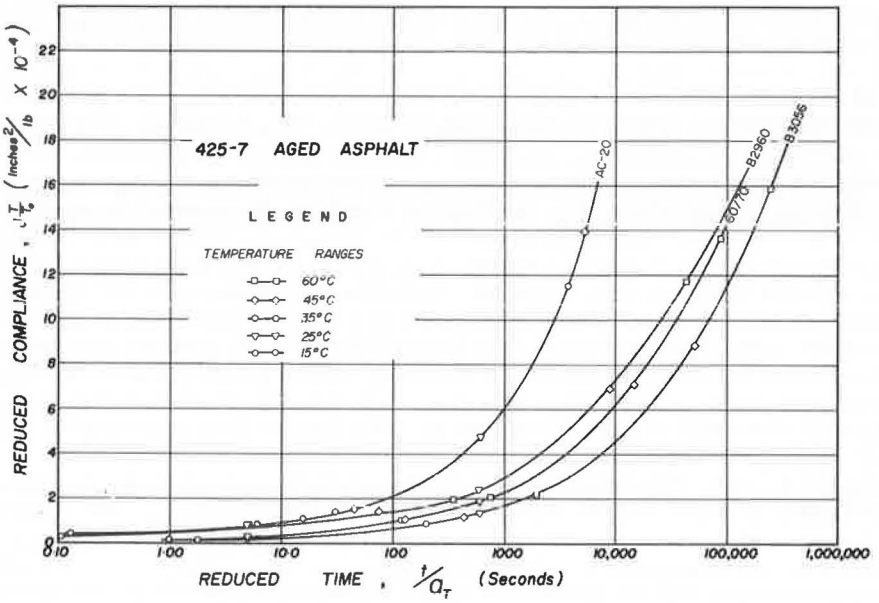


Figure 10. Master creep curves for four asphalt cements of constant aging condition.

In viscoelastic materials such as bituminous mixtures, it is often assumed that the material is thermorheologically a simple system. That is, it is considered that the deformation process is only governed by a single thermally activated process. This assumption simplifies the analysis of creep and flow in rheological bodies.

In this study, the activation energy has been calculated by using two different approaches. In the first method, the temperature-dependence function $a_T$ determined from the superposition of creep curves was used. This function, which is related to the relaxation and retardation processes, can be expressed by

$$a_T = e^{\frac{\Delta E}{R}\left(\frac{1}{T} - \frac{1}{T_o}\right)}$$

where

$T_o$ = reference temperature °K (298 C was used in this study);
$T$ = test temperature °K;
$R$ = universal gas content = 1.986; and
$\Delta E$ = activation energy, cal/mole.

In the second method it is assumed that creep deformation can be expressed by

$$\epsilon = f(\sigma, \theta)$$

where $\sigma$ is the applied stress and $\theta$ is the temperature-dependence term of creep strain given by

$$\theta = \int_o^t \exp\left(\frac{-\Delta E}{RT}\right) dt$$

Assuming that $\Delta E$ is independent of time, then

$$\theta = t \exp\left(\frac{-\Delta E}{RT}\right)$$

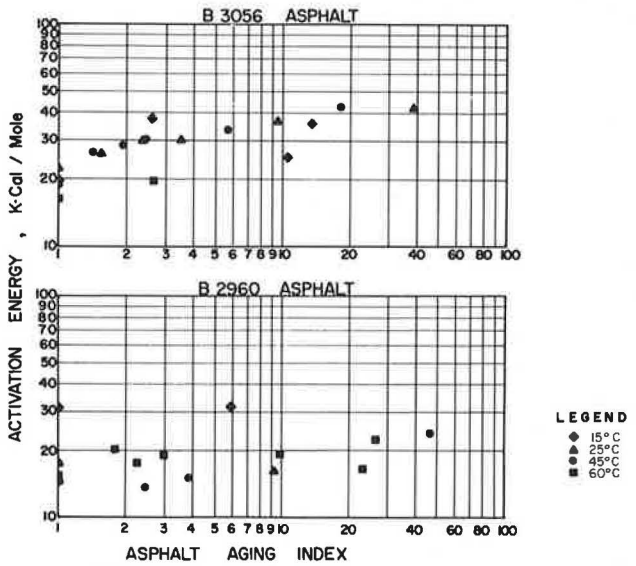


Figure 11. Variation of mixture activation energy with asphalt aging index.

To determine $\Delta E$, creep tests are conducted at various temperatures and the creep compliance, J(t) (strain per unit stress), is plotted against experimental time for various temperatures. Then the experimental times $t_i$ corresponding to a constant compliance at various temperatures $T_i$ can be recorded. According to the equation, for a constant stress and constant compliance, the time $t_i$ and the temperature $T_i$ are related by

$$\theta = t \exp\left(\frac{-\Delta E}{RT}\right) = \text{constant}$$

or

$$t = c \exp\left(\frac{+\Delta E}{RT}\right)$$

That is, when a plot of log $t_i$ and $1/T$ is prepared, the slope of the resulting straight line is equal to $(\Delta E/2.303\ R)$

To show the utility of the activation energy for durability analysis of mixtures, a comparison has been made between the activation energy of mixtures and those of bituminous binders. The results of this comparison have been inconclusive. Furthermore, a plot of mixture activation energy against the asphalt aging index shows that the observed relation is not very promising (Fig. 11). The failure of activation energy to provide a durability criterion for the bituminous mixture can be attributed to many factors. First, the activation energy may not be independent of time and temperature as stated in the theory of rate processes. Second, the aged bituminous mixture may not be controlled by a single thermally activated process; instead, a number of micromechanisms with different activation energies might be involved in the deformation process. Therefore, it is probable that the values reported here are merely a crude approximation of the actual activation energies that contribute to the deformation of aged and unaged specimens.

## SUMMARY AND CONCLUSIONS

In this study the rheological properties of four asphalts at various aging conditions and test temperatures have been investigated to verify previously reported relationships between asphalt aging index and temperature. The creep response of Ottawa sand mixtures at various temperatures and aging conditions has been analyzed, and viscoelastic model parameters and overall response of asphalts at various ages have been compared. The use of the activation energy concept as a tool for durability characterization of asphaltic mixtures has been studied. The following are the major conclusions of this study:

1. The asphalt aging index-temperature relation, when extended to low temperatures, deviates from linearity. This deviation is attributed to the temperature-dependency of asphalt's activation energy at low temperatures.
2. For a constant aging condition, B-2960 asphalt, among the four asphalts, is the most susceptible to aging. Pronounced differences in aging characteristics of other asphalts could be observed by using the aging index as the measure of aging effect on asphalt properties.
3. The viscoelastic parameters calculated for sand-asphalt mixtures are related to the aging index of asphalts. These results indicate that the various viscoelastic parameters obtained from creep are affected by the degree of aging, type of asphalt, and test temperature. Based on the overall creep response, it is also shown that the sand-asphalt mixture becomes stiffer with increasing degrees of aging. The change in mixture stiffness represented by the creep compliance is affected by the type of asphalt, degree of aging, and test temperature.
4. The activation energy of bituminous mixtures has been calculated by using two separate methods. Activation energy does not provide a simple tool for the analysis of aging. It is postulated that the time and temperature dependency of activation energy as well as possibility of involvement of a number of thermally activated processes complicates the interpretation of results.

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# Blending Lightweight Aggregates With Natural Aggregates for the Production of Bituminous Concrete

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This paper describes physical properties of lightweight aggregates, basic design criteria for the inclusion of lightweight aggregates in bituminous mixes, and laboratory test data reflecting indices of strength and performance of bituminous mixes containing lightweight aggregates.

Photographs show the bleb structure (macroscopic and micropscopic) of the lightweight aggregate particles. Physical properties of lightweight aggregates are described with test data (ranges) regarding unit weight, specific gravity, percent absorption, percent abrasion, and degradation under cycles of freezing and thawing.

Basic concepts for bituminous mix design are reviewed and revisions are suggested for the design of bituminous mixes containing blends of lightweight aggregates. The primary revision consists of modifying the design criteria based on weight measurements so as to accurately reflect units of volume for the mix constituents. A simplified procedure is included for the blending of lightweight aggregates with natural aggregates to satisfy a given gradation specification.

The laboratory test data reported consist of Marshall stability values, stabilometer and cohesiometer values, effects of water on cohesion, and air permeability of the compacted mix. The range of values obtained from these tests indicates a high level of performance.

•THE SEARCH for aggregates suitable for the production of bituminous paving mixtures has been greatly intensified by the increase in aggregate consumption and the current requirements for friction-textured aggregates. The increasing demand for coarse aggregates has stimulated research and development relating to production and use of lightweight aggregates. This research has revealed that high-quality lightweight aggregates have proven merit for bituminous pavement construction. However, the use of lightweight aggregate for the design of bituminous mixtures necessitates a departure from the criteria established for the use of aggregates from natural sources.

This paper contains a brief description of research for a further and more effective use of lightweight aggregate for the production of bituminous concrete. The research consisted of a theoretical analysis of basic design criteria and a program of laboratory testing of bituminous mixes containing lightweight aggregates. Test data are discussed and a volume design technique is described.

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Paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction and presented at the 48th Annual Meeting.

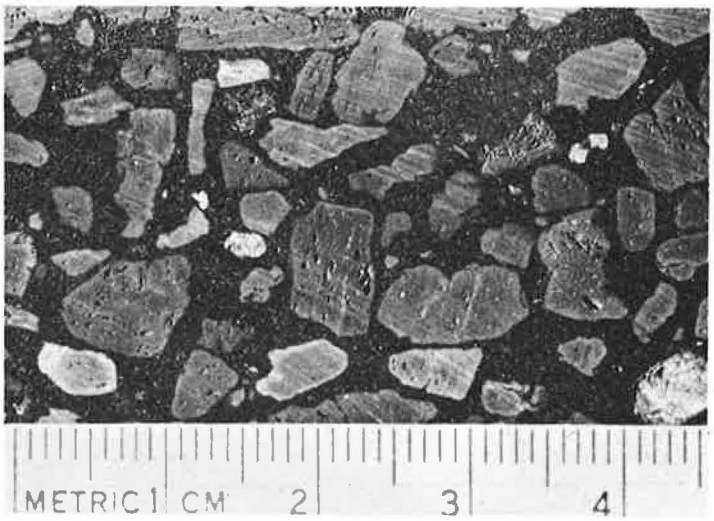


Figure 1. Shell effect and cellular structure of lightweight aggregate in bituminous concrete.

## LIGHTWEIGHT AGGREGATES AND PHYSICAL PROPERTIES

The lightweight aggregates referred to are produced by subjecting raw clay or shale to elevated temperatures in a rotary kiln. During the kiln firing operation, the parent material is expanded (bloated) and hardened. The cellular structure of this bloated material is similar to that found in blast furnace slag. As the particles of raw material reach pyroplastic conditions in the kiln, a shell is formed on the surface and the interior is bloated by the release of gases. Since the cellular air spaces are enclosed in a shell and are not interconnected, the asphalt absorptive characteristics of this type of aggregate are low in view of the total volume of air voids that are within the aggregate particles. The cellular structure also gives the aggregate particles a friction-textured structure that consistently yields a high coefficient of friction under normal abrasive action of automobile traffic. Figure 1 shows the shell effect and the cellular structure of lightweight aggregate particles in a mixture of bituminous concrete. Figure 2


Figure 2. Surface texture of particles of lightweight gate passing the No. 4 sieve.

shows the surface texture of particles of lightweight aggregates passing the No. 4 sieve.

Physical properties of lightweight aggregates that are of primary concern for the design of bituminous mixes are unit weight, specific gravity, percent absorption, percent abrasion, and degradation under cycles of freezing and thawing. To minimize the number of variables, the lightweight aggregate for laboratory testing was limited to one source. Typical ranges in the test data relating to the significant properties of aggregates from this source are as follows:

| | | |
|---|---|---|
| Unit weight | ASTM C-29-67T | 33-43 lb/cu ft |
| Specific gravity | Bryant method (1) | 0.9-1.2 |
| Percent absorption | Bryant method (1) (14 days) | 17-20 percent |
| Percent abrasion | ASTM C-131-66 | 26-31 percent |
| Degradation under cycles of freezing | Gallaway test (2) | 3-8 percent after 50 cycles |

Indices of the physical properties of several other lightweight aggregate sources are reflected in these test data.

## DESIGN CRITERIA FOR BITUMINOUS MIXES CONTAINING LIGHTWEIGHT AGGREGATES

Bituminous mix design consists of a rational determination of the optimum gradation of aggregates and the optimum percentage of asphalt for the production of bituminous concrete with a level of stability and durability commensurate with the needs for performance. The guidelines and design criteria established for the determination of the optimum gradation of aggregates and optimum percentage of asphalt reflect many years of coordinated laboratory and field research work by highway departments and other organizations concerned with the design of bituminous concrete mixes. These design criteria and empirical guidelines are based on the unit weights (weight per unit volume) of typical or natural aggregates. These established weight-volume relationships are distorted when lightweight aggregates are substituted for natural aggregates. Therefore, the established design guidelines must be revised to reflect units of volume when lightweight aggregates are blended with natural aggregates in preparing bituminous mixes. Other considerations relating to the use of lightweight aggregates for hot-mix asphalt pavements were reported by Gallaway and Harper (3).

### Volumetric Analysis

The design criteria for bituminous mixes containing lightweight aggregates must simultaneously reflect an accurate account of the units of volume and the units of weight for each of the materials included in the mixture. A similar analysis of conventional asphaltic concrete mixes is necessary for an accurate account of voids in the asphaltic matrix (4).

The following two formulas are used for simultaneous analyses of the units of volume and units of weight when bituminous mixtures contain aggregates having significant differences in specific gravities:

$$Vmx = Vb + Vna + Vla + Vv \quad (1)$$

$$\gamma mx = \frac{(\gamma b)\ (Vb) + (\gamma na)\ (Vna) + (\gamma la)\ (Vla)}{Vmx} \quad (2)$$

where

Vmx = Volume of bituminous mix,
Vb = Volume of bituminous material (asphalt),

Vna = Volume of natural aggregate,
Vla = Volume of lightweight aggregate,
Vv = Volume of voids,
$\gamma$mx = Unit weight of bituminous mix,
$\gamma$b = Unit weight of bituminous material (asphalt),
$\gamma$na = Unit weight of natural aggregate, and
$\gamma$la = Unit weight of lightweight aggregate.

## Gradation

The optimum gradation of aggregates for a bituminous mixture is normally determined from a series of laboratory tests conducted on mixes prepared from aggregate blends or combinations that hold promise for an economical production of high-quality mixes. A gradation curve is prepared for a graphical analysis of the relative percentages of the aggregate volumes within the various grade fractions. The established methodology for gradation analyses and the blending operation is based on weight measurements of the grade fractions of natural aggregates. The conventional weight measurements of the various grade fractions may be expressed as relative percentages of a total aggregate volume when the aggregates have a common specific gravity (5). However, the weights of grade fractions having different specific gravities must be converted to equivalent volumes in order to make a realistic analysis of the gradation of the aggregate combination or blend.

A simplified procedure is included for blending synthetic aggregates with natural aggregates to obtain a desired gradation. This procedure is for the preparation of a "paper blend" or combination of aggregates by volume and by weight to satisfy a given gradation specification. This volume analysis is also expressed in terms of weight for batching purposes.

## Volumetric Blending to Satisfy Gradation Specification

The procedure described was prepared for the blending of aggregates having different specific gravities. In brief, the procedure consists of a paper analysis of the grade fractions resulting from a trial or assumed paper blend of the aggregates under consideration. The trial blend ratios reflect units of volume. The procedure for blending lightweight aggregates with natural aggregates is as follows:

1. Examine the grade fractions reflected in the available aggregate sources. Materials or material combinations that reflect the desired grade fractions are selected for trial blending analyses.
2. Tabulate gradation data for the aggregates selected on an analysis sheet as shown in Figures 3 and 4 (analysis sheet must reflect percentages passing a specified sieve size and retained on the next smaller sieve size).
3. Select trial blend ratios after making a careful study of gradation specifications and the grade fractions available in the materials selected for blending (computations are simplified by using a total of 10 blend parts).
4. Multiply the grade fraction percentages by the blend ratios selected (Figs. 3, 4).
5. Total the grade fraction percentages in each column and divide by the total number of blend parts.
6. Compare the gradation of the blended aggregate combination with specifications.
7. Repeat steps 3, 4, and 5 until a material combination is obtained to meet specifications. Materials reflecting other grade fractions may be included if necessary.

An example of volumetric blending by trial and error is included for a further explanation of this procedure:

Problem—Determine the blend ratios to satisfy the Asphalt Institute specifications for a Type IVa mix. Volume measurements are required for an accurate analysis of the specified grade fractions, whereas weight measurements are required for accurate batching procedures.

| SPECIFICATIONS AND MATERIALS | BLEND PARTS | GRADE FRACTIONS 1/2" | 3/8" | 4 | 8 | 30 | 50 | 100 | 200 | PAN | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ASPHALT INSTITUTE TYPE IVa MIX | | 20-0 | 25 | 20-25 | 17-21 | 5-6 | 5-7 | 4-6 | 4-10 | TOTAL | |
| SYNTHETIC AGGREGATE MATERIAL - A | | 10 | 40 | 40 | 7 | 3 | – | – | – | 100 | |
| LIMESTONE CHIPS MATERIAL – B | | 10 | 15 | 15 | 50 | 5 | 5 | – | – | 100 | |
| FIELD SAND MATERIAL – C | | – | – | – | – | 15 | 25 | 35 | 25 | 100 | |
| TRIAL NO. 1 | TRY 3 PARTS OF A, 1 PART OF B, AND 1 PART OF C | | | | | | | | | | |
| A | 3 | 30 | 120 | 120 | 21 | 9 | – | – | – | 300 | |
| B | 1 | 10 | 15 | 15 | 50 | 5 | 5 | – | – | 100 | |
| C | 1 | – | – | – | – | 15 | 25 | 35 | 25 | 100 | |
| TOTALS | 5 | 40 | 135 | 135 | 71 | 29 | 30 | 35 | 25 | 500 | |
| DIVIDE BY 5 | 1 | 8 | 27 | 27 | 14 | 6 | 6 | 7 | 5 | 100 | |
| NOTE THE DIFFERENCES BETWEEN ABOVE PERCENTAGES AND SPECIFIED PERCENTAGES | | | | | | | | | | | |
| | | | | | | | | | | | |

*PERCENT PASSING 1/2" AND RETAINED ON 3/8"

Figure 3. Tabular sheet for aggregate blending.

Given—(a) Gradation specifications (limits for Asphalt Institute Type IVa mix (Figs. 3, 4)), (b) grade fractions for one synthetic aggregate and two natural aggregates (Figs. 3, 4), and (c) specific gravities for the three aggregates: aggregate A (lightweight), specific gravity = 1.05; aggregate B (limestone chips), specific gravity = 2.71; and aggregate C (field sand), specific gravity = 2.63.

Figures 3 and 4 show an example of trial analyses and the aggregate blending in order to satisfy gradation specifications. The procedure just outlined was used for the solution of the problem (steps 2 through 6).

The theoretical blending of 3 parts lightweight aggregate with 2 parts limestone chips and one part field sand satisfied the gradation specifications for a Type IVa mix. The blend ratios for volume blending are as follows: (a) lightweight aggregate, 50 percent; (b) limestone chips, 33 percent; and (c) field sand, 17 percent.

| SPECIFICATIONS AND MATERIALS | BLEND PARTS | GRADE FRACTIONS 1/2" | 3/8" | 4 | 8 | 30 | 50 | 100 | 200 | PAN | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| TRIAL NO. 2 | TRY 3 PARTS OF A, 2 PARTS OF B, AND 1 PART OF C | | | | | | | | | | |
| A | 3 | 30 | 120 | 120 | 21 | 9 | – | – | – | 300 | |
| B | 2 | 20 | 30 | 30 | 100 | 10 | 10 | – | – | 200 | |
| C | 1 | – | – | – | – | 15 | 25 | 35 | 25 | 100 | |
| TOTALS | 6 | 50 | 150 | 150 | 121 | 34 | 35 | 35 | 25 | 600 | |
| DIVIDE BY 6 | 1 | 8 | 25 | 25 | 20 | 6 | 6 | 6 | 4 | 100 | |
| NOTE THE ABOVE PERCENTAGES SATISFY ASPHALT INSTITUTE SPECIFICATIONS FOR A TYPE IVa MIX. | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
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Figure 4. Tabular sheet for aggregate blending.

Since the aggregates for bituminous mixes are frequently batched by weight, the ratios for weight blending are determined as follows:

| | | |
|---|---|---|
| 1. | Lightweight aggregate, | 0.50 x 62.4 x 1.05 = 32.8 lb |
| 2. | Limestone chips, | 0.33 x 62.4 x 2.71 = 55.6 lb |
| 3. | Field sand, | 0.17 x 62.4 x 2.63 = 27.9 lb |
| | | Total 116.3 lb |

| | | |
|---|---|---|
| 1. | Lightweight aggregate, | 32.8 ÷ 116.3 = 28 percent |
| 2. | Limestone chips, | 55.6 ÷ 116.3 = 48 percent |
| 3. | Field sand, | 27.9 ÷ 116.9 = 24 percent |

Typical sieve analyses of actual blends of lightweight aggregates and natural aggregates are not suitable for field tests or control of the specified grade fractions. Typical gradation analyses are distorted by the weight measurements of the grade fractions of materials having different specific gravities. A volumetric analysis of the various grade fractions may be used for an approximate control test.

### Asphalt Content

Blending lightweight aggregates with natural aggregates results in a significant reduction in the total weight of the aggregate combination. Therefore, the established guidelines for asphalt (asphalt content based on weight of aggregate combination) are distorted by using lightweight aggregates. In view of this relationship, asphalt content for mixes containing lightweight aggregates should be based on aggregate volume instead of aggregate weight. When lightweight aggregates are used, the relationship of the volume of voids to the total volume of mix is recognized as the only consistent index of the needs for asphalt binder.

## LIGHTWEIGHT AGGREGATE MIXES AND VOLUMETRIC ANALYSES

Blends of lightweight aggregate and natural aggregate offer definite promise for upgrading the quality and performance characteristics of bituminous concrete without a significant increase in the cost of the mix. One of the performance characteristics of primary concern is the dependable and high level of surface friction that is obtained from well-designed blends of lightweight aggregate and natural aggregate (6). Other factors relating to performance and cost are as follows:

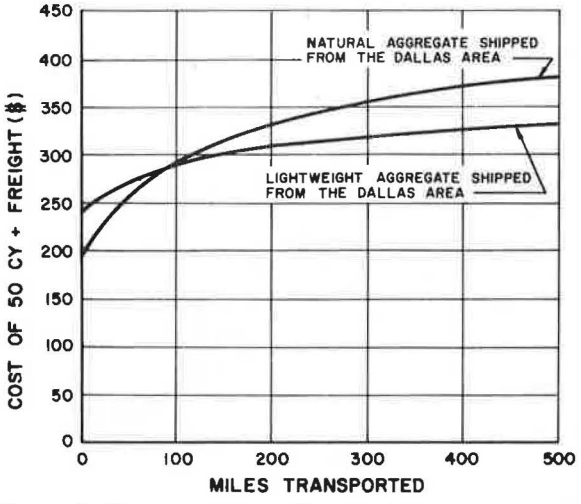


Figure 5. First cost and freight on lightweight and natural aggregates shipped from Dallas, Texas.

1. Lightweight aggregates possess a high angle of internal friction that may be used to improve the stability of a bituminous mixture.
2. Freight or transportation charges (per unit volume) are reduced by the use of lightweight aggregate.
3. Field sand or other locally available materials may be used more extensively for the production of high-stability mixes if blended with lightweight aggregates.

The high angle of internal friction of lightweight aggregate is reflected in the stability values reported in Table 2. The friction and interlocking

characteristics of lightweight aggregates are the properties that upgrade the stability of sand-asphalt mixes. Current costs of natural aggregate and lightweight aggregate (per 50 cu yd from Dallas) are shown in Figure 5.

The bituminous mixtures tested in the laboratory consisted of blends of lightweight aggregate and field sand. The specific gravity of the lightweight aggregate was 1.06 and the gradation consisted of material passing the ½-in. sieve and retained on the No. 30 sieve. The sand had a specific gravity of 2.64 and was graded between the No. 30 and No. 200 sieves. Test data from six of the promising combinations of these two aggregates are included in this report. Blend ratios and asphalt contents for these six aggregate combinations are as follows:

| Mix No. | Lightweight Agg. (percent by vol.) | Field Sand (percent by vol.) | Percent Asphalt (based on weight of agg. comb.) | | |
|---|---|---|---|---|---|
| 1 | 55 | 45 | 6.5, | 7.5, | 8.5 |
| 2 | 60 | 40 | 7.0, | 8.5, | 9.0 |
| 3 | 65 | 35 | 7.5, | 8.5, | 9.5 |
| 4 | 60 | 30 | 8.0, | 9.0, | 10.0 |
| 5 | 75 | 25 | 9.0, | 10.0, | 11.0 |
| 6 | 80 | 20 | 10.0, | 11.0, | 12.0 |

Gradation curves for mixes 1 through 6 are shown in Figure 6. These gradations satisfy the Texas Highway Department specifications for a Type D surface course mix. Data reflecting a volumetric analysis of the constituents of four of these six mixes are

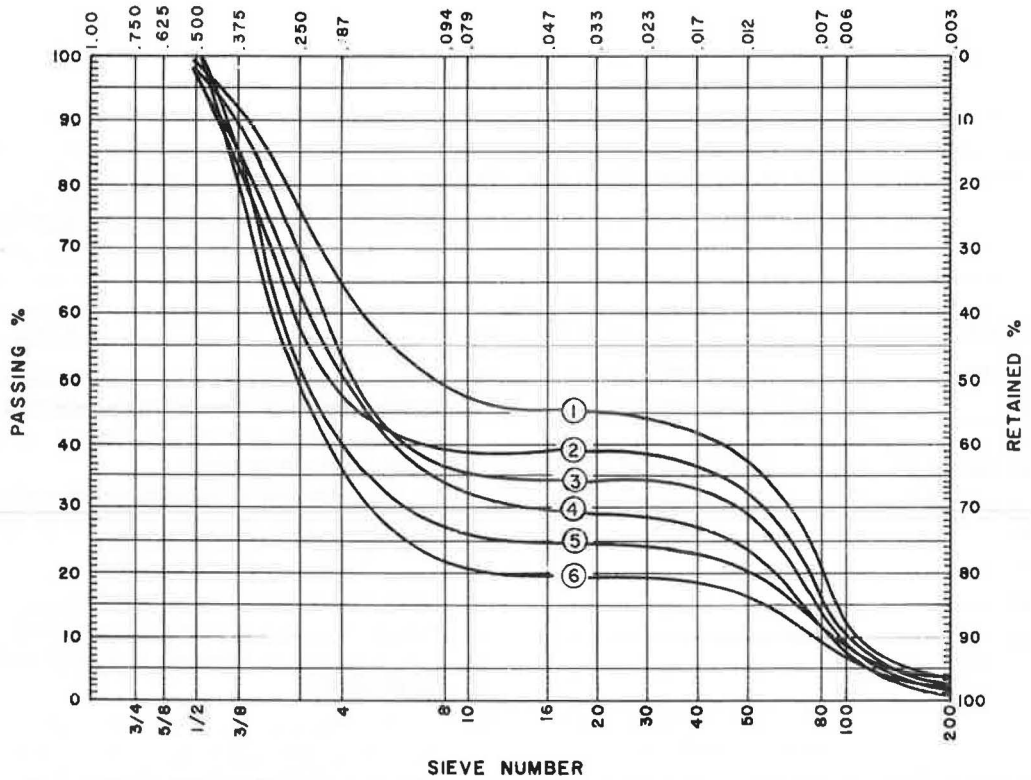


Figure 6. Gradation curves for the six mixes under investigation.

TABLE 1

VOLUMETRIC ANALYSIS OF BITUMINOUS MIXES

| Mix No. | Lightweight Aggregate (percent of agg. comb. by vol.) | Sand (percent of agg. comb. by vol.) | Percent Asphalt (based on weight of agg. comb.) | Lightweight Aggregate (percent by vol. of total mix) | Sand (percent by vol. of total mix) | Asphalt Content (percent by vol. of total mix) |
|---|---|---|---|---|---|---|
| 1 | 55 | 45 | 6.5 | 49.4 | 40.5 | 10.1 |
| 1 | 55 | 45 | 7.5 | 48.7 | 39.8 | 11.5 |
| 1 | 55 | 45 | 8.5 | 47.9 | 39.2 | 12.9 |
| 2 | 60 | 40 | 7.0 | 53.8 | 35.8 | 10.4 |
| 2 | 60 | 40 | 8.0 | 53.0 | 35.3 | 11.7 |
| 2 | 60 | 40 | 9.0 | 52.2 | 34.8 | 13.0 |
| 3 | 65 | 35 | 7.5 | 58.1 | 31.3 | 10.6 |
| 3 | 65 | 35 | 8.5 | 57.3 | 30.9 | 11.8 |
| 3 | 65 | 35 | 9.5 | 56.5 | 30.4 | 13.1 |
| 4 | 70 | 30 | 8.0 | 62.5 | 27.8 | 10.7 |
| 4 | 70 | 30 | 9.0 | 61.7 | 26.4 | 11.9 |
| 4 | 70 | 30 | 10.0 | 60.8 | 26.1 | 13.1 |

included in Table 1. Particular attention is directed to the columns reporting asphalt content by weight and volume.

## LABORATORY TESTING OF BITUMINOUS MIXES

Laboratory testing was conducted to determine the stability and performance characteristics of blends of lightweight aggregate and field sand. Standard test procedures were used to evaluate stability and the effects of water on the bituminous mixtures. The test procedures used for this laboratory investigation are as follows:

1. Marshall stability—ASTM D1559-65;
2. Test for stabilometer value of bituminous mixtures—Texas Highway Department Test Method Tex-208F (7);
3. Cohesiometer value—Test Method Tex-214F (7); and
4. Effect of water on cohesion of compacted bituminous mixtures—ASTM D1075-54.

The other supplementary and control test procedures used for the program of laboratory testing are as follows:

1. Theoretical maximum specific gravity;
2. Specific gravity by the Rice method (8);

TABLE 2

MARSHALL STABILITY—LIGHTWEIGHT AGGREGATE MIXES

| Mix No. | Lightweight Aggregate (percent of agg. comb. by vol.) | Percent Asphalt (based on weight of agg. comb.) | Percent Voids (from Rice Sp. Gr.) | Specific Gravity of Compacted Mix | Plastic Flow (0.01 in.) | Marshall Stability (lb) |
|---|---|---|---|---|---|---|
| 1 | 55 | 6.5 | 11.7 | 1.51 | 9 | 1825 |
| 1 | 55 | 7.5 | 7.3 | 1.53 | 8 | 1283 |
| 1 | 55 | 8.5 | 7.3 | 1.53 | 8 | 1219 |
| 2 | 60 | 7.0 | 11.0 | 1.45 | 6 | 1411 |
| 2 | 60 | 8.0 | 8.1 | 1.48 | 8 | 1132 |
| 2 | 60 | 9.0 | 5.7 | 1.50 | 5 | 948 |
| 3 | 65 | 7.5 | 12.7 | 1.38 | 9 | 2295 |
| 3 | 65 | 8.5 | 9.0 | 1.42 | 9 | 2529 |
| 3 | 65 | 9.5 | 7.7 | 1.44 | 10 | 2704 |
| 4 | 70 | 8.0 | 10.0 | 1.36 | 8 | 2788 |
| 4 | 70 | 9.0 | 6.2 | 1.37 | 7 | 1587 |
| 4 | 70 | 10.0 | 5.5 | 1.38 | 8 | 1729 |
| 5 | 75 | 9.0 | 2.2 | 1.40 | 8 | 3307 |
| 5 | 75 | 10.0 | 3.5 | 1.41 | 13 | 3466 |
| 5 | 75 | 11.0 | 0.6 | 1.45 | 14 | 4055 |
| 6 | 80 | 9.5 | 6.6 | 1.30 | 12 | 2216 |
| 6 | 80 | 10.5 | 4.9 | 1.34 | 13 | 3027 |
| 6 | 80 | 11.5 | 4.9 | 1.34 | 14 | 3118 |

Figure 7. Texas gyratory-shear molding press.

3. Percent voids in the compacted mix (8); and
4. Air permeability of the compacted mix (9).

Mixes 1 through 6 were tested for Marshall stability. The test specimens were compacted with 50 blows on each face. The Marshall stability values and other control test data obtained from the testing are included in Table 2.

Mixes 1 through 4 were tested for stabilometer and cohesiometer values, air permeability, and effects of water on cohesion. Test specimens for these tests were compacted with the Texas gyratory-shear molding press (Fig. 7). Specimens were compacted according to the procedure outlined in test method Tex—206 F (7). Figure 8 shows the testing for stabilometer value according to the prescribed test procedure (7). Table 3 contains the stabilometer values, air permeability, and other control data obtained from the laboratory testing of mixes 1 through 4. The unconfined compression test data reflecting the effects of water on cohesion are summarized in Table 4.

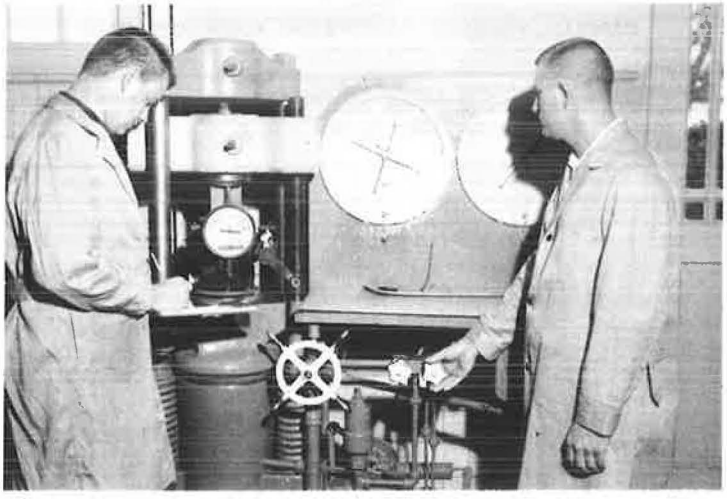

Figure 8. Laboratory testing for stabilometer value.

TABLE 3

STABILOMETER AND SUPPLEMENTARY TEST DATA FOR LIGHTWEIGHT AGGREGATE MIXES

| Mix No. | Lightweight Aggregate (percent of agg. comb. by vol.) | Percent Asphalt (based on weight of agg. comb.) | Bulk Specific Gravity | Specific Gravity (Rice method) | Percent Voids | Air Permeability (ml/sq in./ in. $H_2O$) | Stabilometer Value |
|---|---|---|---|---|---|---|---|
| 1 | 55 | 6.5 | 1.53 | 1.71 | 10.6 | 89.5 | 36.6 |
| 1 | 55 | 7.5 | 1.54 | 1.65 | 6.8 | 33.1 | 36.5 |
| 1 | 55 | 8.5 | 1.58 | 1.65 | 4.3 | 16.9 | 34.5 |
| 2 | 60 | 7.0 | 1.42 | 1.63 | 12.9 | 68.4 | 40.3 |
| 2 | 60 | 8.0 | 1.49 | 1.61 | 7.5 | 27.6 | 36.7 |
| 2 | 60 | 9.0 | 1.51 | 1.59 | 5.0 | — | 40.7 |
| 3 | 65 | 7.5 | 1.38 | 1.58 | 12.7 | 65.1 | 43.2 |
| 3 | 65 | 8.5 | 1.41 | 1.56 | 9.6 | 31.2 | 36.2 |
| 3 | 65 | 9.5 | 1.42 | 1.56 | 8.9 | — | 35.5 |
| 4 | 70 | 8.0 | 1.40 | 1.51 | 7.3 | 89.5 | 48.0 |
| 4 | 70 | 9.0 | 1.43 | 1.46 | 2.1 | 67.6 | 48.0 |
| 4 | 70 | 10.0 | 1.44 | 1.46 | 1.5 | 59.0 | 48.0 |

TABLE 4

UNCONFINED COMPRESSION TESTS FOR EFFECTS OF WATER ON COHESION

| Mix No. | Lightweight Aggregate (percent of agg. comb. by vol.) | Percent Asphalt (based on weight) of agg. comb.) | Bulk Specific Gravity | Unconfined Compressive Strength (lb—dry) | Unconfined Compressive Strength (lb—wet) | Strength Index (percent) |
|---|---|---|---|---|---|---|
| 1 | 55 | 6.5 | 1.48 | 2553 | 2340 | 92 |
| 1 | 55 | 7.5 | 1.49 | 3232 | 2493 | 73 |
| 1 | 55 | 8.5 | 1.51 | 3110 | 2333 | 75 |
| 2 | 60 | 7.0 | 1.42 | 1746 | 1600 | 92 |
| 2 | 60 | 8.0 | 1.45 | 1773 | 1900 | 107 |
| 2 | 60 | 9.0 | 1.47 | 2230 | 2243 | 101 |
| 3 | 65 | 7.5 | 1.38 | 3053 | 2706 | 89 |
| 3 | 65 | 8.5 | 1.41 | 2834 | 2946 | 104 |
| 3 | 65 | 9.5 | 1.42 | 2453 | 2290 | 107 |
| 4 | 70 | 8.0 | 1.37 | 5293 | 4713 | 89 |
| 4 | 70 | 9.0 | 1.36 | 4213 | 4847 | 112 |
| 4 | 70 | 10.0 | 1.37 | 3647 | 4190 | 115 |

## Discussion of Test Data

Mixes 3 through 6 yielded Marshall stability values within the range normally specified for high-stability highway surfaces. All of the mixes yielded stabilometer values above the minimum value of 30 specified by the Texas Highway Department. However, mixes 1 and 4 were the only mixes to satisfy the specifications for a cohesiometer value of 100. The test specimens from mix 1 did show a significant loss in strength when tested for the effects of water on cohesion. This strength loss is attributed primarily to the low asphalt content.

In general, the mixes of lightweight aggregate and sand yielded favorable indices of strength and performance. However, mixes containing more than 70 percent lightweight aggregate reflected some evidence of degradation during compaction. The sand-asphalt matrix (minimum of 30 percent) separates the large particles of lightweight aggregate and cushions the effects of impact compaction.

## SUMMARY

Lightweight aggregates may be blended with natural aggregates to satisfy strict gradation specifications and the current demands for friction-textured highway surfaces. Blending is accomplished by using a trial-and-error paper analysis of the grade fractions reflected in the trial blends or combinations. The blending procedure was pre-

pared specifically for volumetric analyses of the grade fractions of a combination of two or more aggregates having widely different specific gravities.

The use of mixtures of lightweight and normal weight aggregates for the design of bituminous mixes necessitates a departure from the empirical design criteria that have been established for the use of normal-weight aggregates from natural sources. Comprehensive analyses of bituminous mixes containing lightweight aggregates must reflect an accurate account of units of volume and weight for each of the materials included in the mix. Guidelines for asphalt content should be based on voids and total aggregate volume instead of the established procedure that bases asphalt content on the weight of the aggregate combination. The empirical guidelines that have been established for asphalt content (percent by weight of aggregate combination) are distorted by the use of a lightweight aggregate in the mix.

Six mixes or blends of lightweight aggregate and sand yielded promising test data when used for the preparation of bituminous concrete mixes. The test data obtained from Marshall tests, stabilometer tests, cohesiometer tests, and tests for the effects of water on cohesion furnish favorable indices of the structural performance for bituminous concrete mixtures consisting of blends of lightweight aggregate and natural sand. The favorable structural performance and skid resistance obtained from lightweight aggregate mixes point up the potential use of this type of aggregate for the construction of highway surfaces that are in keeping with current needs. The blending procedure described will expedite a rational analysis of economical aggregate combinations that may be used to satisfy gradation specifications and to insure prolonged high skid resistance where lightweight aggregates of the proper quality and amounts are used.

## ACKNOWLEDGMENTS

The authors wish to acknowledge the good judgment and vision of Gifford-Hill and Company, Inc., for their support of the program of research and development reflected in this report.

This research has served not only to answer specific technical questions, but has also furnished financial support and served as an educational medium of several graduate and undergraduate students at Texas A&M University. Without support of this type it would not be possible to carry out our well-rounded program of research in highway safety.

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# Evaluation of Marshall Stability and Flow Values of Asphaltic Paving Mixtures

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This study was conducted to investigate the effects of specimen thickness on both Marshall stability and Marshall flow. Correction factors were derived from regression analyses for both stability and flow, based on data from 18 mixes. Mixtures studied included both original plant mixes and remolded field cores. Thickness of specimens ranged from 0.5 to 3.2 in.

•ONE of the objectives of asphalt paving mixture design is to obtain a mixture with sufficient stability to satisfy, without detrimental distortion or displacement, the service requirements and demands of traffic. Stability of a compacted mixture may be defined as its resistance to such displacement or deformation. In the field, the term implies resistance to shoving and rutting by the action of traffic, and involves resistance to shearing stress. In mixture design, stability and many other mechanical properties have been determined experimentally in the laboratory (rather than calculated from theoretical stress-strain considerations) due to the heterogeneous and viscoelastic nature of the mixture and the complex stress conditions in a flexible pavement system. In the laboratory, stability is expressed in terms of stability tests on compacted mixtures by one or more of several available methods. Criteria have been developed for most of these methods by correlating the results of laboratory tests on compacted paving mixtures with the performance of the paving mixtures under service conditions.

In this approach, it is essential that laboratory methods of sample preparation and testing be able to reproduce, correlate, and predict conditions and behaviors of a mixture in the field during construction and while in service.

Many studies (1-5) have proposed establishing relationships between laboratory specimen compaction and field compaction, and between laboratory specimen density and field pavement density. However, no systematic study on the relationship between stability of laboratory-compacted specimens and stability of the mixture in pavement has been reported. It is often desirable or necessary to be able to evaluate the stability of an asphaltic mixture in a pavement in terms of laboratory design stability. Such an evaluation can provide an engineer the information to readjust laboratory design practice and criteria and field construction practice and control, and to study continuously the factors related to pavement behavior and the effects of time, density, and other environmental factors on stability of mixture in place or in an existing pavement. At present there is no reliable method or information to conduct such an evaluation.

For laboratory analyses there are two logical approaches:

1. Obtain slabs of pavement, remold them into design test specimens in the laboratory, and test for stability; and
2. Cut field cores the size of the design test specimen and test for stability.

Although testing cored samples has not been officially adopted by the ASTM as a standard method in evaluating paving mixtures in place, it is often done to get some information on the in-place mixture in question. This practice is also evidenced by the fact that the current Marshall method adopted by the ASTM actually suggests (6) that, for

Paper sponsored by Committee on Mechanical Properties of Bituminous Paving Mixtures.

core specimens other than 2.5 in., the stability be corrected by multiplying the proper thickness correction factors as suggested by the Corps of Engineers.

In order to make evaluation or meaningful comparison, it is necessary, for either approach, to know the exact effects of specimen thickness on laboratory mixture stability because of the varied pavement thicknesses in place. This study was designed to investigate these effects in terms of the Marshall method.

## MARSHALL FLOW VALUE

The Corps of Engineers (1) recognized that some device for measuring total strain of test specimens would be a valuable addition to the Marshall stability test. A flow meter, which measures the strain occurring in a test specimen between no load and maximum load in hundredths of an inch, was developed. The flow value is a measure of plasticity and flexibility of the compacted mixture and is considered an integrated part of the Marshall method in evaluating the quality of an asphaltic concrete mixture under traffic.

One of the primary factors affecting the flow value is the degree to which the aggregate voids are filled with asphalt. Therefore, for a particular aggregate gradation, the flow value is largely dependent on the asphalt content. A maximum flow value is usually established to prevent the use of excessive asphalt resulting in a plastic mix. A minimum flow value is recommended by the Asphalt Institute because mixes with abnormally low flow values tend to be brittle and less durable.

Goetz (7, 8) and Metcalf (9) have made important studies on the significance of flow value in a Marshall test. Goetz attempted to examine the results of Marshall testing in the light of more fundamental triaxial and unconfined compression tests. Two conclusions from this work are that the Marshall test is a type of confined test due to the curved shape of the testing heads, and that Marshall flow is a measure of internal friction of the compacted mixture. (An inverse linear relationship exists between the internal friction angle and Marshall flow.) By analyzing the stress conditions in a Marshall test and by applying a few assumptions, Metcalf was able to show that the bearing capacity of a paving mixture can be related to Marshall stability and flow by the following approximate form:

$$\text{Bearing capacity (psi)} = \frac{\text{Stability}}{\text{Flow}} \times \frac{(120 - \text{Flow})}{100}$$

The approximate unconfined compressive strength corresponding to a given Marshall result is

$$\frac{\text{Stability}}{100}\left[8 - \tfrac{1}{2}\,\frac{(30 - \text{Flow})^2}{10}\right]$$

More significantly, Metcalf showed that bearing capacities calculated from the Marshall test generally agreed with the performance of the test sections, thus demonstrating the importance of the flow value in the paving mixture design and especially in the evaluation of in-place properties of mixtures in pavement. It was again shown that the ability of the Marshall test (and the bearing capacity calculated from the Marshall test) to satisfactorily predict field performance depended largely on whether or not laboratory-prepared specimens could reproduce or be related to the properties of in-place material in the pavement.

The effect of sample thickness on flow value was studied within a limited range in the original studies of the Corps of Engineers. It was concluded that specimen thickness has little effect on flow, and thus no correction factors were considered necessary for flow value measured for specimens that were not of a standard 2.5-in. thickness. No published data of systematic study support or disprove this, although it was suspected by some that flow might be influenced by the thickness of the specimen. The Corps of Engineers conclusion was based on observations on specimens thicker than 1.5 in. Consequently, a study on specimen thickness effect on Marshall flow, especially for samples thinner than 1.5 in., is needed to properly evaluate Marshall test results on thinner

specimens. This is especially desirable because thin hot-mix wearing surfacings are becoming more popular with highway engineers both in the United States and abroad.

## MATERIALS AND METHODS

Materials used in this study were obtained in the field from three associated research projects conducted at the Bituminous Research Laboratory, Iowa State University, during the 1964, 1967, and 1968 construction seasons. Mixes were taken at the plants during the regular paving construction projects in Iowa. Four-in. cores were taken immediately after compaction for mixes No. 1 to 9. Cores were taken after 1 year of service from mixes No. 10 to 16. Core thicknesses varied from 0.6 in. to 2.5 in. Gradation and percent bitumen of these mixes are given in Table 1.

Plant mixes were heated in a laboratory oven to between 250 and 275 F and compacted to various thicknesses according to the standard Marshall method (50 blows on each side). Compacted specimens included duplicate samples of standard thickness (2.5 in.) and thicknesses equal to those of the pavement cores in which the mixes were placed.

Standard Marshall stability and flow at 140 F were determined for all samples after sample thickness was measured and bulk specific gravity was determined.

After testing, both laboratory-compacted specimens prepared from original plant mixes and field cores were remolded (by heating in the oven to 250 F) and compacted by the standard Marshall method. These were again tested for thickness, bulk specific gravity, Marshall stability, and flow.

## RESULTS

Results of thickness effect study on mixes No. 1 to 9 are given in Table 2. Related information for mixes No. 10 to 16 is given in Table 3. Each set of data represents the average of at least two identical specimens.

### Stability vs Specimen Thickness

The relationship between specimen thickness and stability at 140 F was studied by regression analysis for all mixes that were compacted by the standard Marshall method (50 blows per side) and that had more than three thicknesses. Correlation coefficients were also calculated for each mix. Linear equations of regression, sample size n, and correlation coefficient r are given in Table 4(a). For comparison, the equations of regression were also obtained by using the Corps of Engineers correction factors for standard stability of 500, 1000, and 2000 lb, designated in the table as CE 500, CE 1000,

TABLE 1

GRADATION AND BITUMEN CONTENT OF MIXES STUDIED

| Property | Mix No. | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 1 | 2 | 3 | 4 | 6 | 7 | 8 | 9 |
| Percent passing sieve | | | | | | | | | | | | | | | |
| 1 in. | 100 | 100 | 100 | 100 | 100 | 100 | 100 | | | | | | | | 100 |
| 3/4 in. | 100 | 100 | 100 | 100 | 100 | 99 | 100 | | | | | 100 | | | |
| 1/2 in. | 97 | 91 | 95 | 93 | 96 | 91 | 92 | | | | | 94 | | | 78 |
| 3/8 in. | 86 | 77 | 79 | 78 | 81 | 78 | 77 | 100 | 100 | 100 | 100 | 77 | 100 | 100 | 62 |
| No. 4 | 65 | 56 | 59 | 57 | 63 | 60 | 59 | 81 | 89 | 92 | 78 | 60 | 85 | 85 | 49 |
| No. 8 | 51 | 44 | 46 | 45 | 52 | 48 | 47 | 62 | 67 | 58 | 59 | 43 | 67 | 63 | |
| No. 16 | 36 | 37 | 33 | 34 | 41 | 34 | | 44 | | | | | | | |
| No. 30 | 28 | 32 | 25 | 27 | 31 | 27 | 28 | 30 | 34 | 34 | 31 | 25 | 35 | 35 | 23 |
| No. 50 | 16 | 21 | 17 | 19 | 18 | 17 | 19 | 16 | 21 | 23 | 18 | 19 | 25 | 25 | |
| No. 100 | 8 | 10 | 12 | 12 | 12 | 9 | 12 | 12 | 13 | 13 | 13 | 9 | | 13 | |
| No. 200 | 5.7 | 7.7 | 7.8 | 8.0 | 8.5 | 7.2 | 7.0 | 9.3 | 9.9 | 9.4 | 9.4 | 7.1 | 8.4 | 8.9 | 6.7 |
| Percent bitumen | 5.3 | 5.7 | 5.5 | 5.3 | 4.8 | 4.3 | 4.8 | 7.5 | 6.3 | 6.3 | 7.5 | 5.0 | 7.0 | 5.3 | 5.5 |
| Year | 1964 | 1964 | 1964 | 1964 | 1964 | 1964 | 1964 | 1967 | 1967 | 1967 | 1967 | 1968 | 1968 | 1968 | 1968 |

TABLE 2

EFFECT OF THICKNESS ON STABILITY, FLOW, AND BEARING CAPACITY OF MIXES NO. 1 TO 9

| Mix No. | Type of Mix | Percent Bitumen | Thickness (in.) | Specific Gravity | Stability (lb) | Flow (0.01 in.) | Bearing Capacity (psi) | Corrected Bearing Capacity (psi) |
|---|---|---|---|---|---|---|---|---|
| 1 | P | 7.5 | 0.79 | 2.35 | 400 | 5 | 92.0 | 291.1 |
| | | | 0.91 | 2.34 | 490 | 4 | 142.1 | 390.4 |
| | | | 1.11 | 2.33 | 690 | 4 | 200.1 | 450.7 |
| | | | 1.27 | 2.28 | 520 | 5 | 119.0 | 235.4 |
| | | | 1.36 | 2.32 | 840 | 4 | 243.6 | 447.8 |
| | | | 1.52 | 2.24 | 750 | 7 | 121.1 | 199.1 |
| | | | 1.61 | 2.35 | 1250 | 5 | 287.5 | 446.4 |
| | | | 2.12 | 2.34 | 1800 | 7 | 296.6 | 342.7 |
| | | | 2.54 | 2.31 | 2060 | 7 | 332.5 | 327.3 |
| 2 | P | 6.3 | 0.93 | 2.25 | 600 | 5 | 138.0 | 370.9 |
| | | | 1.09 | 2.20 | 570 | 6 | 108.3 | 248.4 |
| | | | 2.46 | 2.19 | 1920 | 16 | 124.8 | 126.8 |
| | | | 2.60 | 2.18 | 2000 | 15 | 140.0 | 134.6 |
| | | | 2.93 | 2.07 | 2350 | 11 | 232.9 | 198.7 |
| | CR-O | 6.3 | 0.94 | 2.25 | 460 | 6 | 87.4 | 232.5 |
| | | | 1.04 | 2.28 | 660 | 5 | 151.8 | 364.9 |
| | | | 2.41 | 2.21 | 1650 | 10 | 181.5 | 188.3 |
| | | | 2.50 | 2.24 | 2200 | 8 | 308.0 | 308.0 |
| | | | 2.65 | 2.22 | 2100 | 10 | 231.0 | 217.9 |
| 3 | P | 6.3 | 0.96 | 2.280 | 400 | 5 | 92.0 | 239.6 |
| | | | 1.03 | 2.292 | 500 | 5 | 115.0 | 279.1 |
| | | | 1.22 | 2.260 | 520 | 5 | 119.6 | 245.1 |
| | | | 1.31 | 2.271 | 650 | 5 | 149.5 | 285.3 |
| | | | 1.46 | 2.282 | 850 | 5 | 195.5 | 343.8 |
| | | | 1.54 | 2.302 | 1080 | 5 | 248.4 | 403.3 |
| | | | 2.51 | 2.288 | 2020 | 9 | 249.1 | 248.1 |
| | CR-O | 6.3 | 0.89 | 2.324 | 490 | 5 | 112.7 | 316.6 |
| | | | 1.03 | 2.298 | 520 | 5 | 119.6 | 290.3 |
| | | | 1.13 | 2.342 | 760 | 6 | 144.4 | 319.5 |
| | | | 1.28 | 2.332 | 880 | 7 | 142.0 | 277.5 |
| | | | 2.57 | 2.322 | 2390 | 9 | 294.8 | 286.7 |
| 4 | P | 7.5 | 0.99 | 2.239 | 580 | 6 | 109.2 | 278.3 |
| | | | 1.14 | 2.278 | 1380 | 6 | 262.2 | 574.9 |
| | | | 2.46 | 2.223 | 2710 | 10 | 298.1 | 302.9 |
| | | | 2.74 | 2.206 | 2830 | 14 | 214.3 | 195.5 |
| | P + 1 | 7.5 | 0.81 | 2.302 | 650 | 6 | 122.6 | 381.2 |
| | | | 1.13 | 2.258 | 850 | 6 | 161.5 | 357.3 |
| | | | 2.53 | 2.230 | 2550 | 12 | 229.5 | 226.8 |
| | CR-O | 7.5 | 0.89 | 2.345 | 980 | 6 | 185.3 | 523.0 |
| | | | 1.12 | 2.296 | 1150 | 6 | 218.5 | 487.7 |
| | | | 1.25 | 2.282 | 980 | 6 | 186.2 | 372.8 |
| | | | 1.40 | 2.268 | 1090 | 6 | 206.2 | 369.8 |
| 4 | CR-O | 7.5 | 1.55 | 2.258 | 1180 | 7 | 199.7 | 307.2 |
| | | | 1.70 | 2.247 | 1200 | 7 | 228.0 | 284.9 |
| | | | 2.65 | 2.242 | 2720 | 10 | 298.7 | 282.3 |
| 6 | P | 5.0 | 1.44 | 2.40 | 1035 | 6 | 196.6 | 341.4 |
| | | | 1.58 | 2.39 | 960 | 6 | 182.4 | 288.6 |
| | | | 1.70 | 2.41 | 1285 | 6 | 244.1 | 359.0 |
| | | | 1.72 | 2.40 | 1270 | 6 | 241.3 | 350.7 |
| | | | 1.73 | 2.37 | 1160 | 6 | 220.4 | 318.5 |
| | | | 1.83 | 2.40 | 1590 | 6 | 302.1 | 412.7 |
| | | | 1.98 | 2.39 | 1500 | 6 | 285.0 | 359.9 |
| | | | 2.13 | 2.38 | 1440 | 7 | 232.4 | 272.8 |
| | | | 2.39 | 2.39 | 1990 | 9 | 245.4 | 256.7 |
| 7 | P | 7.0 | 0.59 | 2.32 | 240 | 4 | 69.6 | 294.9 |
| | | | 0.71 | 2.31 | 260 | 4 | 75.4 | 265.5 |
| | | | 0.72 | 2.31 | 270 | 4 | 78.3 | 271.9 |
| | | | 0.86 | 2.29 | 215 | 4 | 62.4 | 181.3 |
| | | | 0.98 | 2.29 | 335 | 4 | 97.2 | 247.8 |
| | | | 1.11 | 2.28 | 365 | 4 | 105.8 | 238.4 |
| | | | 1.25 | 2.27 | 370 | 5 | 85.1 | 170.2 |
| | | | 1.38 | 2.28 | 495 | 4 | 143.6 | 260.1 |
| | | | 1.52 | 2.28 | 540 | 4 | 156.6 | 257.6 |
| | | | 1.64 | 2.28 | 570 | 5 | 131.1 | 199.9 |
| | | | 2.14 | 2.31 | 1275 | 6 | 242.2 | 283.0 |
| 8 | P | 5.3 | 0.54 | 2.33 | 270 | 6 | 51.3 | 237.5 |
| | | | 0.69 | 2.29 | 260 | 6 | 49.4 | 178.9 |
| | | | 0.81 | 2.31 | 345 | 5 | 79.4 | 244.9 |
| | | | 0.93 | 2.32 | 440 | 6 | 83.6 | 224.7 |
| | | | 1.05 | 2.33 | 590 | 6 | 112.1 | 266.9 |
| | | | 1.18 | 2.34 | 610 | 6 | 115.9 | 245.6 |
| | | | 1.30 | 2.33 | 735 | 7 | 117.0 | 225.1 |
| | | | 1.43 | 2.34 | 935 | 7 | 150.9 | 263.9 |
| | | | 1.56 | 2.34 | 1065 | 6 | 202.4 | 324.3 |
| | | | 2.22 | 2.33 | 1710 | 7 | 276.0 | 310.9 |
| | CR-O | 5.3 | 0.54 | 2.33 | 235 | 6 | 44.6 | 206.7 |
| | | | 0.69 | 2.32 | 290 | 6 | 55.1 | 199.6 |
| | | | 0.80 | 2.33 | 345 | 7 | 55.7 | 174.0 |
| | | | 0.92 | 2.33 | 415 | 6 | 78.8 | 214.3 |
| | | | 2.43 | 2.33 | 1805 | 8 | 252.7 | 259.9 |
| 9 | P | 5.5 | 1.12 | 2.37 | 570 | 6 | 108.3 | 241.7 |
| | | | 1.28 | 2.39 | 760 | 6 | 144.4 | 282.0 |
| | | | 1.78 | 2.37 | 1120 | 6 | 212.8 | 298.9 |
| | | | 1.87 | 2.39 | 1375 | 6 | 261.2 | 349.3 |
| | | | 2.16 | 2.36 | 1240 | 7 | 200.2 | 231.7 |
| | | | 2.43 | 2.37 | 1860 | 7 | 300.3 | 308.1 |

CR-O Remolded O-month core  P Original plant mix  P + 1 One hour after leaving plant

TABLE 3

PROPERTIES OF MIXES NO. 10 TO NO. 16

| Mix No. | Type of Mix | Thick-ness (in.) | Specific Gravity | Stability (lb) | Flow (0.01 in.) | Bearing Capacity (psi) | Corrected Bearing Capacity (psi) |
|---|---|---|---|---|---|---|---|
| 10 | P | 2.50 | 2.36 | 1510 | 12 | 135.9 | 135.9 |
| | | 2.94 | 2.36 | 1700 | 14 | 128.9 | 109.5 |
| | | 3.19 | 2.38 | 1950 | 16 | 126.8 | 99.3 |
| 11 | P | 2.56 | 2.32 | 2490 | 9 | 307.1 | 299.9 |
| | | 2.63 | 2.33 | 2650 | 11 | 262.6 | 249.6 |
| | | 2.81 | 2.31 | 3020 | 19 | 160.5 | 142.8 |
| | | 3.13 | 2.32 | 3240 | 22 | 144.3 | 115.3 |
| 12 | P | 1.31 | 2.35 | 980 | 8 | 137.2 | 261.8 |
| | | 1.50 | 2.34 | 1200 | 9 | 148.0 | 246.7 |
| | | 1.81 | 2.30 | 1280 | 12 | 115.2 | 159.1 |
| | | 1.94 | 2.30 | 1500 | 11 | 148.6 | 191.5 |
| | | 2.13 | 2.31 | 1860 | 10 | 204.6 | 240.1 |
| | | 2.56 | 2.33 | 2540 | 13 | 209.1 | 204.2 |
| 13 | P | 2.50 | 2.40 | 1570 | 8 | 219.8 | 219.8 |
| | CR-12 | 2.13 | 2.36 | 1650 | 10 | 181.5 | 213.0 |
| | | 2.25 | 2.37 | 1770 | 10 | 194.7 | 216.3 |
| | | 2.50 | 2.35 | 1950 | 11 | 193.2 | 193.2 |
| | | 2.56 | 2.36 | 2050 | 12 | 184.5 | 180.2 |
| 14 | P | 2.50 | 2.26 | 1330 | 8 | 172.2 | 172.2 |
| | CR-12 | 1.44 | 2.28 | 300 | 7 | 48.4 | 84.1 |
| | | 1.50 | 2.30 | 340 | 12 | 30.6 | 51.0 |
| | | 1.56 | 2.29 | 380 | 13 | 31.3 | 50.1 |
| 15 | P | 2.50 | 2.40 | 1400 | 10 | 154.0 | 154.0 |
| | CR-12 | 2.00 | 2.45 | 680 | 15 | 47.6 | 59.5 |
| | | 2.06 | 2.45 | 770 | 17 | 46.6 | 56.6 |
| | | 2.13 | 2.45 | 930 | 18 | 52.7 | 61.9 |
| | | 2.50 | — | 1230* | 18 | 69.7 | 69.7 |
| 16 | P | 2.50 | 2.32 | 1440 | 9 | 177.6 | 177.6 |
| | CR-12 | 2.50 | 2.36 | 1240 | 20 | 62.0 | 62.0 |
| | | 2.56 | 2.36 | 1480 | 21 | 69.8 | 68.4 |
| | | 2.63 | 2.36 | 1510 | 20 | 75.5 | 71.8 |
| | | 2.69 | 2.35 | 1680 | 24 | 67.2 | 62.5 |

P Original plant mix
CR-12 Remolded 12-month core

*Corrected stability

TABLE 4

EFFECT OF THICKNESS

(a) Thickness t vs Stability S

| Mix No. and Type | Equation | Sample Size n | Correlation Coefficient r | Standard-Size Specimen Stability So |
|---|---|---|---|---|
| 1 P | S = 1019 t - 520 = 0.495 Sot - 0.253 So | 9 | 0.9622* | 2060 |
| 2 P | S = 907 t - 328 = 0.463 Sot - 0.167 So | 5 | 0.9969* | 1960 |
| 2 CR-O | S = 939 t - 378 = 0.427 Sot - 0.172 So | 5 | 0.9779* | 2200 |
| 3 P | S = 1071 - 674 = 0.530 Sot - 0.307 So | 7 | 0.9897* | 2020 |
| 3 CR-O | S = 1158 t - 590 = 0.503 Sot - 0.257 So | 5 | 0.9977* | 2300 |
| 4 P | S = 1174 - 276 = 0.433 Sot - 0.102 So | 4 | 0.9688* | 2710 |
| 4 P + 1 | S = 1138 t - 345 = 0.446 Sot - 0.135 So | 3 | 0.9968* | 2550 |
| 4 CR-O | S = 1003 t - 184 = 0.371 Sot - 0.068 So | 7 | 0.9226* | 2700 |
| 6 P | S = 977 t - 433 = 0.489 Sot - 0.217 So | 9 | 0.9035* | 2000 |
| 7 P | S = 577 t - 228 = 0.444 Sot - 0.175 So | 11 | 0.9063* | 1300** |
| 8 P | S = 901 t - 358 = 0.475 Sot - 0.191 So | 10 | 0.9879* | 1900** |
| 8 CR-O | S = 863 t - 311 = 0.469 Sot - 0.169 So | 5 | 0.9964* | 1840 |
| 9 P | S = 868 t - 385 = 0.482 Sot - 0.214 So | 6 | 0.9499* | 1800 |
| 10 P | S = 614 t - 47 = 0.407 Sot - 0.031 So | 3 | 0.9722 | 1510 |
| 11 P | S = 1291 t - 742 = 0.518 Sot - 0.298 So | 4 | 0.9609* | 2490 |
| 12 P | S = 1222 t - 731 = 0.481 Sot - 0.288 So | 6 | 0.9675* | 2540 |
| 13 CR-12 | S = 873 t - 206 = 0.448 Sot - 0.106 So | 4 | 0.9932* | 1950 |
| 14 C-12 | S = 544 t - 476 = 0.641 Sot - 0.560 So | 3 | 0.9997* | 850 |
| 15 CR-12 | S = 1933 t - 395 = 1.172 Sot - 1.936 So | 4 | 0.9935 | 1650** |
| 16 CR-12 | S = 2090 t - 3047 = 1.713 Sot - 3.234 So | 4 | 0.9537* | 1240 |
| CE 500 | S = 296 t - 242 = 0.592 Sot - 0.484 So | 15 | 0.9966* | 500 |
| CE 1000 | S = 592 t - 484 = 0.592 Sot - 0.484 So | 15 | 0.9966* | 1000 |
| CE 2000 | S = 1185 t - 968 = 0.592 Sot - 0.484 So | 15 | 0.9966* | 2000 |

(b) Thickness t vs Flow F

| Mix No. and Type | Equation | Sample Size n | Correlation Coefficient r | Standard-Size Specimen Flow Fo |
|---|---|---|---|---|
| 1 P | F = 1.79 t + 2.7 = 0.256 Fot + 0.386 Fo | 9 | 0.7652* | 7 |
| 2 P | F = 4.60 t + 1.4 | 5 | 0.8438 | — |
| 2 CR-O | F = 2.47 t + 3.1 = 0.309 Fot + 0.388 Fo | 5 | 0.9138* | 8 |
| 3 P | F = 2.66 t - 1.7 = 0.296 Fot - 0.196 Fo | 7 | 0.9144* | 9 |
| 3 CR-O | F = 2.33 t + 3.2 = 0.259 Fot + 0.356 Fo | 5 | 0.9465* | 9 |
| 4 P | F = 4.06 t + 1.6 = 0.406 Fot + 0.160 Fo | 4 | 0.9494* | 10 |
| 4 P + 1 | F = 3.73 t + 2.4 | 3 | 0.0845 | |
| 4 CR-O | F = 2.46 t + 3.2 = 0.246 Fo + 0.320 Fo | 7 | 0.9574* | 10 |
| 6 P | F = 2.89 t + 1.1 = 0.361 Fot + 0.137 Fo | 9 | 0.8319* | 8 |
| 7 P | F = 1.12 t + 3.1 = 0.187 Fot + 0.500 Fo | 11 | 0.7802* | 6 |
| 8 P | F = 0.83 t + 5.2 = 0.119 Fot + 0.743 Fo | 10 | 0.6435* | 7 |
| 8 CR-O | F = 1.03 t + 5.5 = 0.129 Fot + 0.688 Fo | 5 | 0.8830* | 8 |
| 9 P | F = 0.83 t + 4.9 | 6 | 0.8048 | — |
| 10 P | F = 5.65 t - 2.3 | 3 | 0.9876 | — |
| 11 P | F = 23.10 t - 49.0 | 4 | 0.9422 | — |
| 12 P | F = 3.49 t + 3.9 = 0.268 Fot + 0.300 Fo | 6 | 0.8344* | 13 |
| 13 CR-12 | F = 4.33 t + 0.5 | 4 | 0.9225 | — |
| 14 CR-12 | F = 5.09 t - 2.9 | 4 | 0.7077 | — |
| 15 CR-12 | F = 22.83 t - 30.4 | 3 | 0.9726 | — |
| 16 CR-12 | F = 16.83 t - 22.4 | 4 | 0.7349 | — |

TABLE 4 (Continued)

(c) Thickness t vs Bearing Capacity B

| Mix No. and Type | Equation | Sample Size n | Correlation Coefficient r |
|---|---|---|---|
| 1 P | B = 129.59 t + 13.3 | 9 | 0.8168* |
| 2 P | B = 32.35 t + 84.0 | 5 | 0.6134 |
| 2 CR-O | B = 80.35 t + 38.6 | 5 | 0.8149 |
| 3 P | B = 102.9 t + 19.6 | 7 | 0.8270* |
| 3 CR-O | B = 109.89 t + 11.1 | 5 | 0.9951* |
| 4 P | B = 44.86 t + 138.7 | 4 | 0.4898 |
| 4 P + 1 | B = 58.06 t + 84.7 | 3 | 0.9816 |
| 4 CR-O | B = 62.65 t + 121.6 | 7 | 0.8933* |
| 6 P | B = 59.06 t + 130.6 | 9 | 0.4528 |
| 7 P | B = 103.77 t - 8.3 | 11 | 0.9242* |
| 8 P | B = 140.66 t - 40.9 | 10 | 0.9796* |
| 8 CR-O | B = 113.63 t - 24.9 | 5 | 0.9971* |
| 9 P | B = 126.38 t - 19.6 | 6 | 0.8911* |
| 10 P | B = -13.55 t + 169.4 | 3 | -0.9861 |
| 11 P | B = -277.02 t + 989.4 | 4 | -0.8937 |
| 12 P | B = 64.40 t + 39.7 | 6 | 0.7598 |
| 13 CR-12 | B = 6.31 t + 173.6 | 4 | 0.1987 |
| 14 CR-12 | B = 22.39 t + 3.6 | 4 | 0.7540 |
| 15 CR-12 | B = 40.55 t - 34.7 | 3 | 0.8065 |
| 16 CR-12 | B = 33.83 t - 19.2 | 4 | 0.4980 |

(d) Thickness t vs Corrected Bearing Capacity B

| Mix No. and Type | Equation | Sample Size n | Correlation Coefficient r |
|---|---|---|---|
| 1 P | B = -11.68 t + 365.0 | 9 | -0.7010* |
| 2 P | B = -88.35 t + 392.8 | 5 | -0.8163 |
| 2 CR-O | B = -36.29 t + 331.6 | 5 | -0.4227 |
| 3 P | B = 4.09 t + 284.9 | 7 | 0.0356 |
| 3 CR-O | B = -12.49 t + 315.3 | 5 | -0.4517 |
| 4 P | B = -110.16 t + 539.8 | 4 | -0.5994 |
| 4 P + 1 | B = -90.81 t + 457.1 | 3 | -0.9994* |
| 4 CR-O | B = -133.12 t + 576.2 | 7 | -0.7864* |
| 6 P | B = -77.51 t + 471.0 | 9 | -0.4563 |
| 7 P | B = -6.11 t + 249.9 | 11 | -0.0694 |
| 8 P | B = 65.03 t + 176.1 | 10 | 0.7534* |
| 8 CR-O | B = 34.89 t + 173.4 | 5 | 0.8575 |
| 9 P | B = 24.25 t + 242.3 | 6 | 0.2781 |
| 10 P | B = -53.81 t + 269.7 | 3 | -0.9955* |
| 11 P | B = -312.95 t + 1072.7 | 4 | -0.9120 |
| 12 P | B = -37.14 t + 286.9 | 6 | -0.4269 |
| 13 CR-12 | B = -77.94 t + 384.6 | 4 | -0.9305 |
| 14 CR-12 | B = -3.86 t + 68.2 | 4 | -0.1152 |
| 15 CR-12 | B = 23.53 t + 10.8 | 4 | 0.3422 |
| 16 CR-12 | B = 8.28 t + 44.6 | 4 | 0.1454 |

*Significant at 5 percent level
**Estimated stability

So = Stability for standard size 2.5-in. specimen
Fo = Flow value for standard size 2.5-in. specimen

and CE 2000. Since the equations are functions of standard stability (stability of standard 2.5-in. specimen), they were also expressed in terms of standard stability So in the following form:

$$S = a\,Sot + b\,So$$

where

S = stability of specimen with thickness t, lb;
So = stability of 2.5-in. specimen, lb;
t = thickness of specimen, in.; and
a, b = constants (a being the slope, b the y intercept).

The following can be noted:

1. Stability is significantly correlated with thickness of specimen for all mixes over the thickness range studied (i. e., from 0.5 in. to 3.2 in.) regardless of the type of mix, asphalt content, aggregate gradation, cores or remolded cores, as long as the compactive efforts are the same.

2. The relation between stability and thickness is linear. When expressed in terms of standard stability So, the relation can be stated as

$$S = a\,Sot - b\,So$$

Constant a varied from 0.37 to 1.71 and constant b varied from 0.03 to 3.23. If core samples are disregarded due to small sample size and possible variation due to coring, the ranges of a and b are narrowed to between 0.37 and 0.53 for a and between 0.07 and 0.31 for b for those mixes whose correlation coefficients are significant at the 5 percent level. The averages of all these mixes are 0.47 for a and 0.20 for b, compared with the constants derived from the Corps of Engineers data of 0.59 and 0.48 respectively. Assuming that the relationship is linear over the range of thickness of 0.5 in. to 3.5 in., the comparsion in Table 5 can be made between this study and the Corps of Engineers data for a mixture with a standard stability of 1000 lb.

From Table 5 it is obvious that the Corps of Engineers correction factors cannot be used for specimens or cores less than 1 in. thick, which are often used for thin hot-mix wearing courses.

## Flow vs Specimen Thickness

The relationship between Marshall flow and specimen thickness in terms of equations of regression and correlation coefficients is given in Table 4(b). Except for core samples, coefficients were positive and significant for essentially all mixes regardless of type of mix and asphalt content. The relation was linear—an increase in thickness resulted in higher flow value for the same mix and compactive effort. The table also shows the flow value F at thickness t as a function of standard-size specimen flow Fo for all the mixes that showed significant correlation at the 5 percent level. The average equation relating F, Fo, and t for all mixes is

$$F = 0.26\,Fot + 0.35\,Fo$$

For a mix with a flow of 10 at 2.5 in., a difference of 0.5 in. in specimen thickness will mean a difference of one unit in flow. As an approximation the correction factors for flow for specimens other than 2.5 in. thick should be

| Thickness (t) | Correction Factor | Thickness (t) | Correction Factor |
|---|---|---|---|
| 0.5 | 2.08 | 2.5 | 1.00 |
| 1.0 | 1.64 | 3.0 | 0.88 |
| 1.5 | 1.35 | 3.5 | 0.79 |
| 2.0 | 1.15 | | |

The Corps of Engineers recommendation that flow correction is not necessary is essentially correct within a narrow range of thickness variation, since the flow change is not sensitive to small thickness change.

## Bearing Capacity vs Specimen Thickness

It has been repeatedly pointed out by many that the Marshall stability alone cannot adequately measure the ability of a paving mixture to resist displacement under load. Bearing capacities, as suggested by Metcalf (9), were calculated for all specimens and mixes because they offer means of evaluating Marshall stability and flow jointly. The relationship between specimen thickness and bearing capacity, as calculated directly from stability S and flow F of specimens of varied thickness [$B = S/F \times (120 - F)/100$], is given in Table 4(c). The relationship between thickness and bearing capacity when thickness is taken into consideration [$B = S/tF \times (120 - F)/140$] (termed as corrected bearing capacity) is given in Table 4(d).

TABLE 5

COMPARISON OF DATA

| Thickness t (in.) | Iowa State University S = So(0.47 t - 0.20) = 470 t - 200 | | Corps of Engineers S = So(0.59 t - 0.48) = 590 t - 480 | |
|---|---|---|---|---|
| | Stability S (lb) | Correction Factor | Stability S (lb)* | Correction Factor* |
| 0.5 | 35 | 28.60 | -165 (—) | — (—) |
| 1.0 | 270 | 3.70 | 110 (180) | 9.10 (5.56) |
| 1.5 | 505 | 1.98 | 405 (450) | 2.47 (2.78) |
| 2.0 | 740 | 1.35 | 700 (680) | 1.43 (1.47) |
| 2.5 | 980 | 1.02 | 1000 (1000) | 1.00 (1.00) |
| 3.0 | 1210 | 0.83 | 1290 (1320) | 0.78 (0.76) |
| 3.5 | 1450 | 0.69 | 1590 (—) | 0.63 (—) |

*Figures in parentheses were obtained by using the recommended correction factors rather than from the regression equation

Eight of the 20 mixes indicated significant correlation between thickness and uncorrected bearing capacity, all positive. This is to be expected since the stability increases with increasing sample thickness. However, no general equation can be established either as a function of thickness t or in terms of thickness t and standard bearing capacity Bo.

Theoretically the corrected bearing capacity of a mixture should be independent of specimen thickness. Data in Table 2 and the correlation coefficients in Table 4(d) generally support this. The fact that 5 of the 20 mixes showed significant correlation could be attributed to: (a) the assumption that the stability is directly proportional to specimen thickness in corrected bearing capacity calculation, and (b) measurement error or repeatability of the stability test.

## CONCLUSIONS

Within the limits of this study, the following conclusions can be drawn:

1. Both Marshall stability and flow should be corrected to properly evaluate and compare specimens of nonuniform and nonstandard thicknesses. The Corps of Engineers correction factors are not adequate for thin specimens.
2. Further research is needed to establish criteria for evaluating results of stability tests on core specimens and evaluating flow values of specimens other than standard size.

## ACKNOWLEDGMENT

The author wishes to thank Professors Ladis H. Csanyi and Robert M. Nady, both of the Department of Civil Engineering, Iowa State University, for reviewing the manuscript, and Messrs. Ken Bettis and Dennis Tripp for most of the testings.

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# Orientation of Particles With Special Reference To Bituminous Paving Materials

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A method is described for measuring particle orientation in bituminous mixtures and for analyzing the data by statistical techniques to determine the existence and degree of preferred orientation. Samples prepared by alternative laboratory compactive procedures are compared with each other and with the compactive effort of a field roller in respect to the degree of preferred orientation. The conclusion is drawn that for the particular mix examined given number of passes of a field roller can be equated with a certain number of passes of a laboratory roller that achieves the same degree of orientation. This structural element of the field specimen is confirmed to be more closely simulated by using the laboratory roller for specimen compaction than by using static compaction. The relationship between preferred orientation and sample anisotropy as revealed by strength tests in mutually perpendicular directions is discussed.

•THE importance of orientation of particles and its correlation to the macroproperty of the assembly of particles has been recognized by geologists since the 1930's. The existence of preferred orientation of pebbles in coarse-grained unconsolidated deposits has been demonstrated by Wadell (1) and Krumbein (2). Recently, Gipson (3) measured the particle orientation of shale and related the orientation to fissile planes.

However, in the field of highway engineering materials very little has been done with regard to measurement of orientation of particles, although several researchers have recognized its importance. Lee and Markwick (4) stated: "It is desirable to imitate the method of consolidation used on the road so that there should be similarity in aggregate orientation and in density variation." Goetz (5) stated: "Density is only one criterion of identity and differences caused by such factors as orientation of aggregate particles, non-uniform density in the specimen, aggregate breakage in the mould are to be recognized too." Finally, Hennes and Wang (6) accepted the existence of preferred particle orientation in bituminous materials without actually measuring it quantitatively or proving its existence.

## ORIENTATION AS AN ELEMENT OF STRUCTURE

The work of Frederick (7) reveals that in order to describe a state of packing of an assembly of particles it is necessary to know for every particle in the mass: (a) its size, (b) its shape, (c) its location in space, and (d) its orientation in space. If these

Paper sponsored by Committee on Mechanical Properties of Bituminous Paving Mixtures.

factors can be defined in numerical terms, then it is possible to describe the state of packing of an assembly of n particles by an array of vectors in a matrix form:

$$\begin{array}{cccc} X_1\ Y_1\ Z_1 & D_1 & F_1\ S_1\ R_1/A_1\ T_1 & P_1\ Q_1\ r_1 \\ X_2\ Y_2\ Z_2 & D_2 & F_2\ S_2\ R_2/A_2\ T_2 & P_2\ Q_2\ r_2 \\ \vdots & \vdots & \vdots & \vdots \\ X_i\ Y_i\ Z_i & D_i & F_i\ S_i\ R_i/A_i\ T_i & P_i\ Q_i\ r_i \end{array}$$

In this matrix, $X_i$, $Y_i$, and $Z_i$ describe the location of the ith particle in space and $D_i$ describes its size. $F_i$ and $S_i$ describe the three-dimensional shape of the ith particle and $R_i$, $A_i$, $T_i$ describe details of its surface form. Finally, the factors $P_i$, $Q_i$ and $r_i$ describe the orientation of the ith particle in space (in terms of compass direction, dip, and end orientation, as defined next).

The size of a particle can be numerically described by the diameter of a sphere of the same volume. The shape of a particle is one of its most complicated attributes. Methods exist for expressing quantitatively such shape characteristics as roundness and sphericity (8), shape factor (9), angularity (10), and surface texture (11). These terms are defined and the relationship between them discussed by Lees (12). In the matrix, sphericity ($S_i$) and shape factor ($F_i$) describe the overall three-dimensional shape. Roundness ($R_i$) and angularity ($A_i$) are alternative shape parameters describing the form of external contour superimposed on the overall shape; surface texture ($T_i$) is available, where needed, to describe the microroughness of the surface. In order to describe the position of a particle in space, the most logical approach is to define the position of the centroid of the particle with respect to some origin. This was done by Bernal (13) for equal-size ideally spherical particles. The only element in the matrix that lacks a rigorous definition and has not previously been described numerically is the orientation of particles.

If one employs the matrix for describing the state of packing of an assembly of equal-size ideally spherical particles, the problem is relatively easy. It is so because, in such an assembly, the elements descriptive of particle size ($D_i$) and shape ($S_i$, $R_i$, $A_i$, and $T_i$) are constant and there is no particle orientation. Therefore, the only variable element of the structure matrix is the location of particles ($X_i$, $Y_i$, and $Z_i$). However, more complications are faced when the matrix is employed for bituminous materials. Bitumnous materials consist of stone particles of various size (ranging from 2 microns to one inch or even larger), shapes, and orientations that are bound together with a binder. It is clear that in such a case none of the elements of the structure matrix is constant.

## METHODS OF ORIENTATION ANALYSIS

In describing the orientation of coarse unconsolidated clastic sediments, Krumbein (2) presents a method of orientation analysis that requires manipulation of individual particles With finer particles, however, manipulation becomes difficult, precluding the application of Krumbein's method for bound road materials where the problem would be to dissolve away the binder without disturbing the position of the particles. Nevertheless, one might be able to employ and/or extend it for orientation anaylsis of water-bound road materials.

Several researchers, in measuring particle orientation in road materials, have adopted a qualitative approach (14). However, the authors are aware of only one project in which an attempt was made, ostensibly, to evaluate the orientation of particles quantitatively. This was done by Puzinauskas in 1964 (15). Puzinauskas' method is

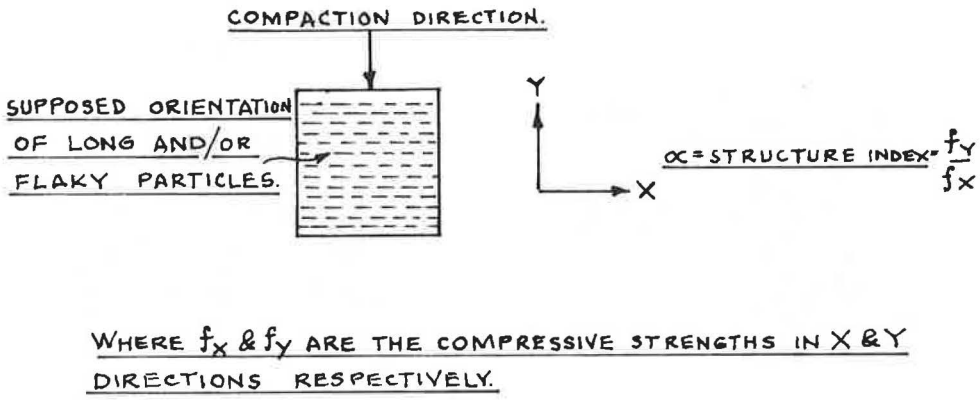


Figure 1. Measurement of structure index.

briefly discussed and it is shown that it does not necessarily provide a measure of the orientation of particles.

Puzinauskas measured the compressive strength of cube-shaped specimens of asphaltic concrete in directions normal and parallel to the direction of compactive effort, calculating a "structure index" as a measure of particle orientation from the results obtained. This structure index is determined as the ratio of the compressive strength measured in the direction parallel to the direction of compactive effort to the compressive strength measured in the direction normal to the direction of compactive effort (Fig. 1). He then suggested that if the structure index is equal to unity, it means the particles are oriented randomly (Fig. 2a). If the structure index were larger than unity, it was taken as an indication that the particles exhibited a state of preferred orientation in the direction normal to the direction of compactive effort (Fig. 2b). Finally, if the structure index were smaller than unity, this indicated that there existed a preferred orientation in the direction parallel to the direction of compactive effort (Fig. 2c).

In the authors' opinion Puzinauskas' structure index does not necessarily measure the orientation pattern of particles. It is in fact a measure of the degree of anisotropy of the specimen rather than a direct measure of particle orientation.

It is clear that ideally spherical particles can have no orientation. Therefore, if Puzinauskas' structure index really measures the orientation of particles, its value for a specimen of ideally spherical particles should be equal to unity. This is clearly not the case because it is apparent that even ideally spherical particles can exhibit either regular or irregular packing structures that are not spherically symmetrical and therefore might be expected to exhibit anisotropic behavior in mechanical tests. Such arrangements might have structure index values bigger or smaller than unity, depending

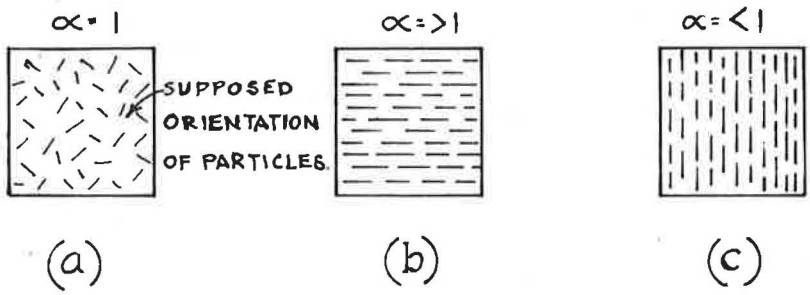


Figure 2. Puzinauskas' method for determining the orientation pattern of a specimen of asphaltic concrete.

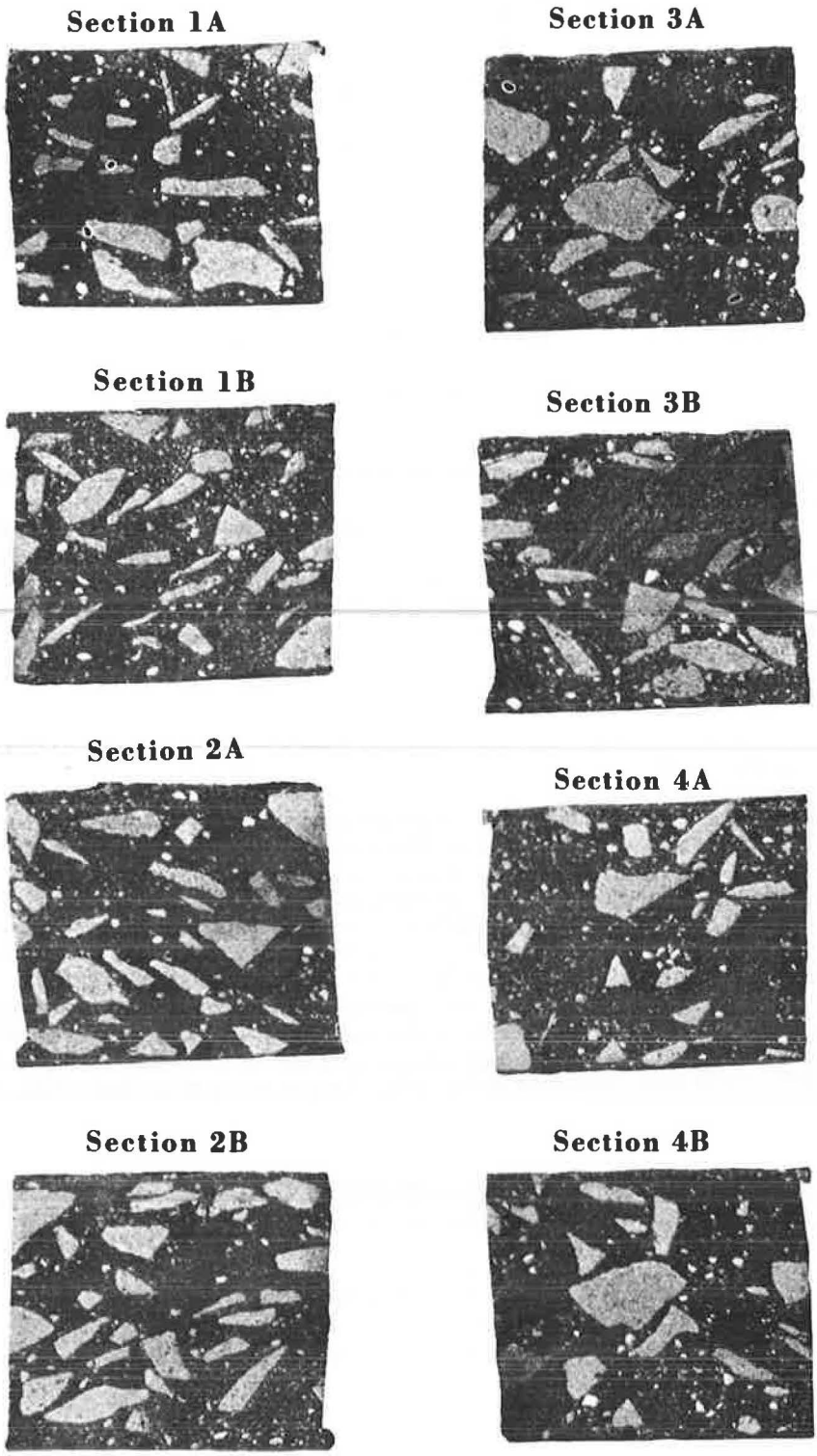


Figure 3. Some of the sections employed for orientation analysis.

on the orientation of packing structure (16). In other words, Puzinauskas' structure index fails to differentiate between the particle orientation and packing orientation; packing orientation would include void orientation.

## METHOD DEVELOPED FOR ORIENTATION ANALYSIS

### Principles of Method

The method developed and employed here for orientation analysis is derived and modified from a method used by Gipson (3) for orientation analysis of sedimentary materials. The method is divided into two parts: (a) the measurement of the orientation directions of individual particles, and (b) the synthesis of the data and measurement of the orientation pattern of all the particles in the assembly.

Determining the orientation of individual particles is based upon examining the projections of the particles visible on a given plane. However, the study of the orientation of individual particles in isolation is of little significance in a system complicated by particles of a wide range of size, shape, and orientations. Statistical analysis is the best means for synthesizing the data and, moreover, statistical methods provide criteria for finding whether the orientation distributions are due to chance or follow some particular law, i. e., whether they exhibit random or preferred orientation.

The method is, at present, two-dimensional. It is simple in calculation and, so far as the investigation has been conducted, reveals the nature of orientation patterns. However, if necessary, the method can be extended to a three-dimensional analysis, and the analysis and the orientation of particles can be obtained by determining their projections on mutually perpendicular planes.

An advantage of this method is that it does not involve removing particles from their original positions. In other words, the particles are not manipulated and hence there is no error or difficulty in orientation measurement of smaller particles. For fine particles, one need only magnify the projections of the particles to the required dimensions for measurement. The method can be directly used for orientation anaylsis of all bound granular construction materials. Moreover, it can be extended to measure the orientation pattern of unbound granular materials. In this case, one should use a suitable solidifying agent as a binder and then the procedure is exactly the same as described.

### Measurement of Orientation Direction of Individual Particles

Step one of the method is to find the projections of the particles on a given plane. For this purpose, the specimen whose orientation pattern is being investigated is cut into thin sections. The thickness of each section should not be less than the maximum size of the aggregate used in the specimen. The sections were obtained by using a diamond-saw cutting machine. The surfaces of the sections were then ground, first using a coarse No. 80 and then a fine No. 400 silicon carbide powder. Grinding was continued until sufficient visual contrast was obtained between the projections of the particles and the matrix. From each specimen 8 inches long an average of 10 sections were obtained. Figure 3 shows examples of some of the sections cut.

The orientation directions of the projections of the particles ($\theta$) visible on the sections were then determined by finding the direction of the longest line that can be drawn on each particle projection (Fig. 4). This constitutes the long dimension method for measurement of particle orientation. Two other methods are also recognized for the measurement of orientation (17). They are (a) the least projection method and (b) the center of area method. In the least projection method, elongation direction is the direction of the two parallel lines with the minimum amount of separation that can be drawn tangent to the particle projection. In the center of area method, elongation direction is the direction of the longest straight line that can be drawn through the center of area of the projection. The center of area is considered to be a two-dimensional equivalent of the center of mass that is the point about which the particle pivots when suspended in a fluid. These methods might each be considered in some respects to be fundamentally more accurate than the long dimension method. For the purposes of this

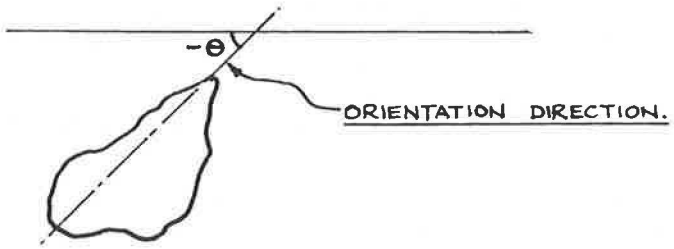


Figure 4. Long dimension method for the measurement of particle orientation.

research, however, the long dimension method was employed because it is the least time-consuming of the methods, and it was felt that no great inaccuracy would result from its use.

The orientation directions should be measured in a particular direction. For this analysis, clockwise direction is chosen as positive and counterclockwise as negative and the angle that the elongation direction of the projection of each particle makes with the horizontal is measured.

Some cubical projections of particles that have no apparent definite orientation direction appear on the sections. These are either originally cubical particles, or long or flaky particles that are cut during the preparation of the sections to give equidimensional cross sections. These projections are not considered in the orientation analysis. Moreover, in samples prepared in the laboratory, some particles seem to be oriented nearly vertically close to the sides of the molds. These projections also are not considered in the analysis because their orientation has clearly been affected by the sides of the moulds, making them unrepresentative of field conditions.

Some further points should be mentioned regarding the measurement of elongation direction (by the long dimension method, for example) as an indication of orientation direction of individual particles. If one is studying the flow of a liquid through the particles, the measurement of elongation direction will not be sufficient for a complete analysis of orientation. In this case, it is also necessary to determine the "end position" of individual particles. Figure 5 shows two particles of the same elongation direction but different end positions.

Because there is no continuous directional flow of any liquid through the particle mass, however, end orientation of the particles does not occur in bituminous materials. In other words, for bituminous materials the measurement of elongation direction, as described and measured by the long dimension method, is considered to be sufficient.

## Measurement of Orientation Pattern—Statistical Analysis

Once the orientation direction of individual particles ($\theta$) has been mesured, synthesis of the data is achieved by employing statistical methods. The size of the sample, i. e., the number of particles to be analyzed, for sufficient accuracy is measured and the frequency distribution of the orientation directions of each specimen is plotted. The next problem is to test the hypothesis that the particles exhibit a preferred orientation. If so, then the mean orientation direction is computed.

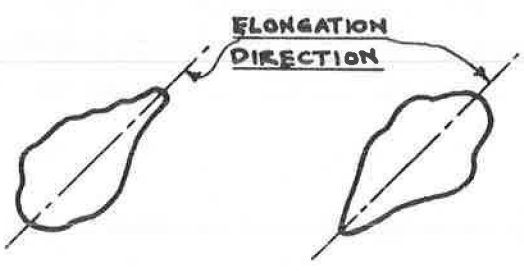


Figure 5. End position as an element of orientation direction.

As used here, the term "preferred orientation" refers to an arrangement of particles that can be statistically shown to exhibit a significant deviation from random orientation. The particles are said

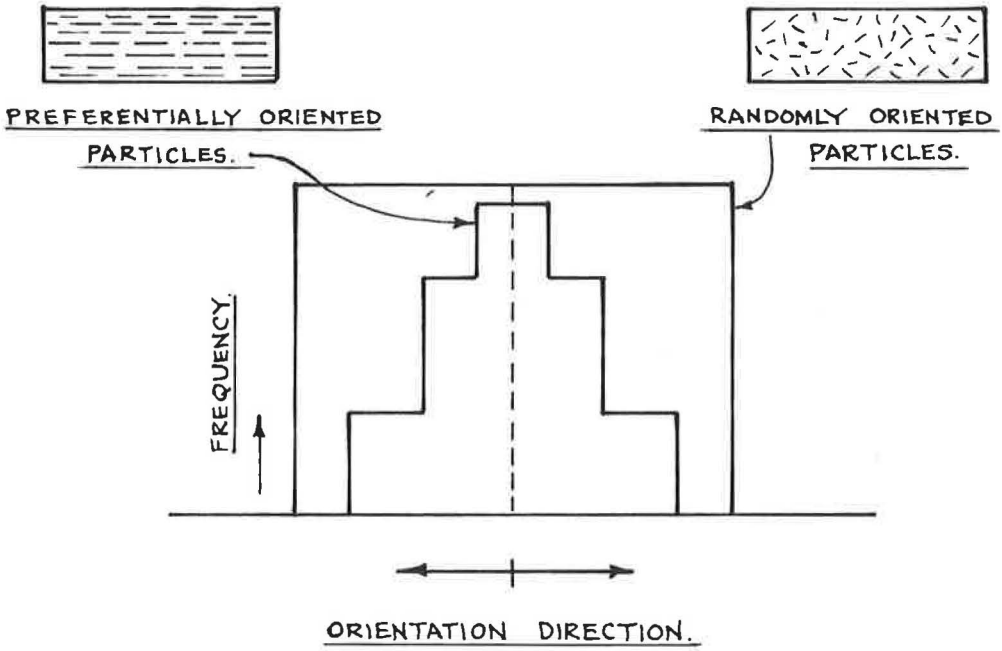


Figure 6. Random and preferred orientation.

to be randomly oriented when the observed frequency distribution of the orientation is rectangular (Fig. 6).

The simplest way of testing this hypothesis is to use fundamental laws of statistics. For this research, a special $\chi^2$ test of significance, developed by Tukey (18), has been employed. Although Tukey did not introduce the statistical justification of his method, a derivation of it was discussed by Middleton (19).

Tukey's method determines whether the frequency distributions obtained for orientation directions are due to natural factors or represent merely chance deviations from random distribution. For this purpose the theory of sampling has been employed, and the level of significance adopted for the test was 0.05 percent. If the calculated probability is less than 0.05, the deviation from chance is considered significant (preferred orientation). On the other hand, if the probability is greater than 0.05, the deviation is not regarded as significant and may be accepted as being due to chance (random orientation).

## MATERIAL, MIX DESIGN, AND MIXING PROCEDURE

The materials selected for coarse and fine aggregate were Rowley Regis basalt and Leighton Buzzard sand respectively. Ordinary portland cement was used as filler. The specific gravities of coarse aggregate, fine aggregate, and filler were 2.85, 2.65, and 3.15 respectively. The binder used was the bitumen "Mexphalte," produced by Shell-Mex and B. P. with 50/60 penetration grade. The specific gravity of the bitumen was measured according to I. P. 59/57 as 1.026 at 25 C.

In order to emphasize orientation, the coarse aggregate particles were all "flaky" in shape. This was done by separation of flaky from non-flaky particles of the coarse aggregate. For this purpose a "shape separator" was constructed operating on the principles discussed by Dunagan (20) and further described by Salehi (21). By using this instrument, separation according to elongation ratio is eliminated; i. e., equi-

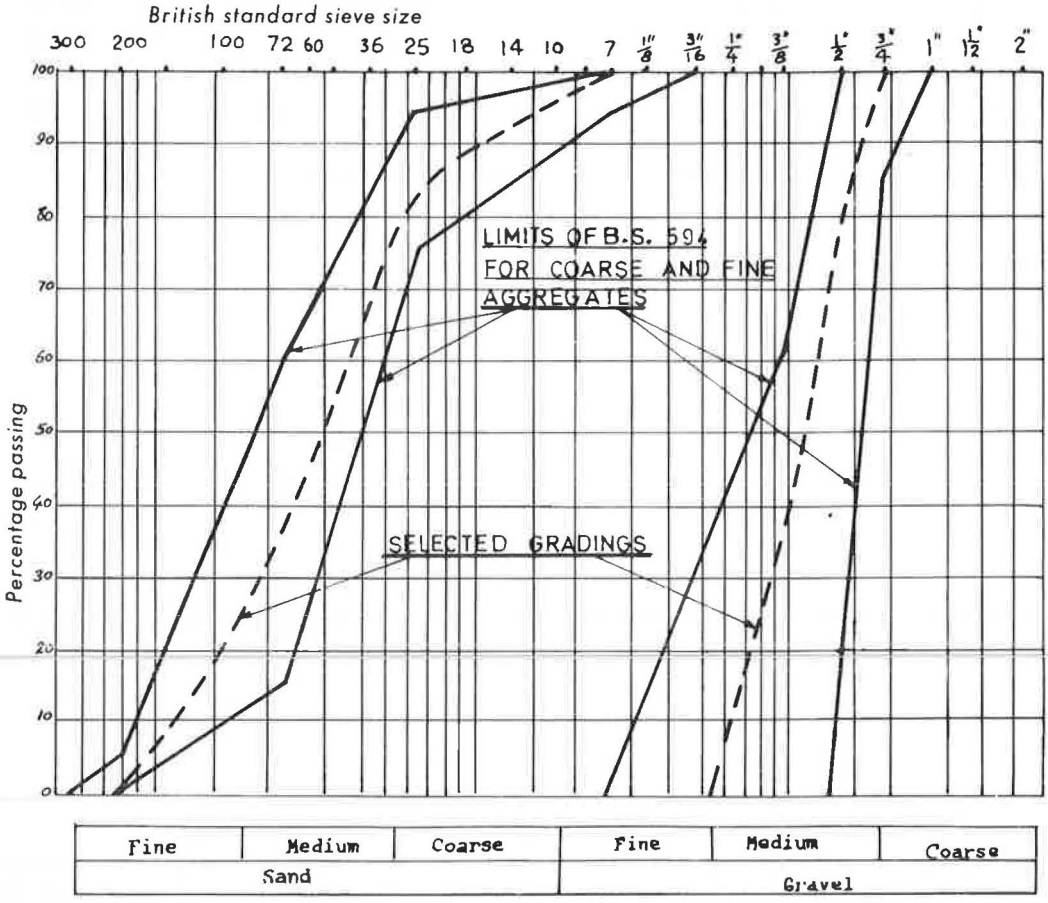


Figure 7. Gradings selected for coarse and fine aggregate.

dimensionals and roads are combined as non-flaky while discs and blades are combined as flaky. *

The gradings selected for coarse and fine aggregate are shown in Figure 7. The mix was designed according to table 7, schedule 1 (wearing course mixture—rock aggregate) of British Standard 594 (22), selecting 35 percent coarse aggregate content. The mixing procedure was that recommended by the Asphalt Institute (23).

## COMPACTION METHODS

### Field Compaction

The mixture was prepared in the laboratory and then transported to the site in an insulated box. Because of the profound influence of compaction temperature on the properties of bituminous materials, every effort was made to maintain the temperature sufficiently high while transporting the mixture to the site. The mixture was then laid

*These terms were defined by Lees (12). It must be appreciated that the use of this instrument, while enormously speeding up the process of shape separation, cannot produce samples with 100 percent accuracy; i.e., the non-flaky fraction will always contain some flaky particles and vice versa. In this work an optimum slope for equivalent accuracy of both fractions was chosen. This slope was 30 deg and the accuracy 84 percent.

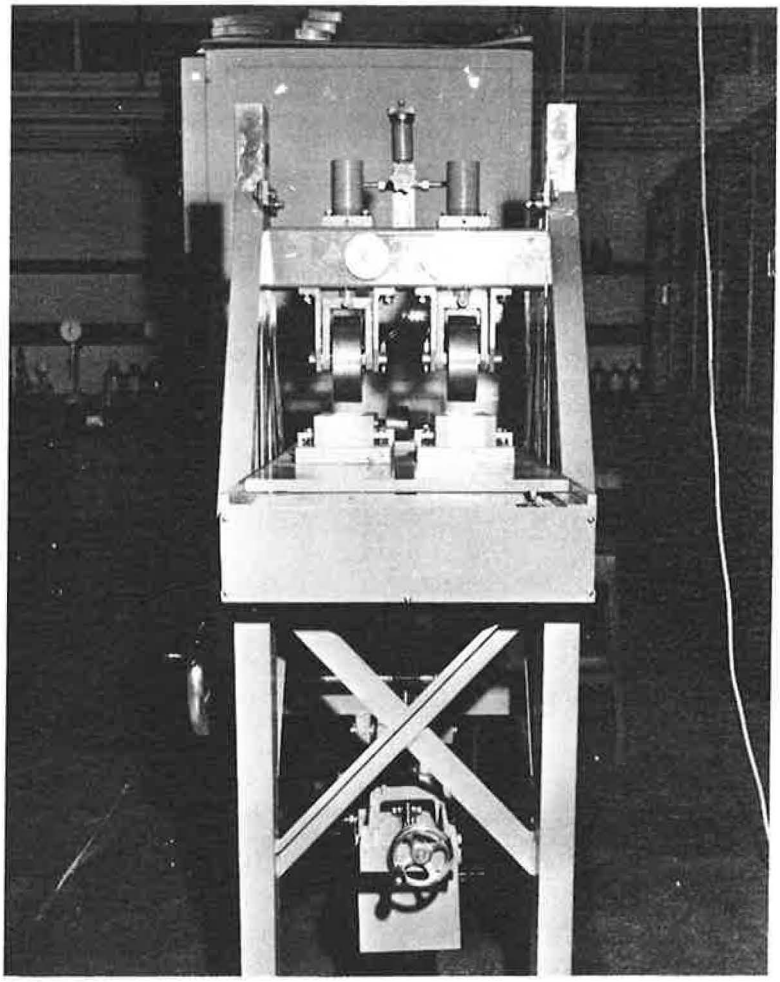

Figure 8. Laboratory roller compaction machine.

as wearing course in 18-in. square patches and compacted by 7 passes of a 6-ton steel-wheel roller at approximately 2 mph. The site was Nechell's Parkway, Birmingham, and the compaction was conducted in collaboration with Tarmac Roadstone Ltd. The compacted slab was carefully cut out by a drill hammer after being allowed to cool and set for 3 days. The slab was then transported back to the laboratory, cut into beam specimens of 8 by 2 by 1.5 in., subsequently to be cut again into the sections required for orientation analysis.

## Laboratory Rolling Compaction

Beam specimens of 8 by 2 by 1.5 in. were prepared in the laboratory by a new laboratory roller compaction machine, a description of which is given by Salehi (21) and shown in Figure 8. During compaction the temperature of the mix was not allowed to fall below 145 C. The roller compaction machine has been designed with the aim of producing realistic specimens. As used in this context the term "realistic specimens" means those that are similar to field specimens of the same material under a given compactive effort with regard to density, degradation, and orientation of particles in the immediate post-rolling condition. It is recognized that subsequent/progressive variations in all three of these properties may take place during the working life of the pavement material. It is intended in a later paper to deal with the density and degradation aspects of specimen structure.

## Laboratory Static Compaction

Beam specimens of 8 by 2 by 1.5 in. were prepared in the laboratory by employing a static maximum compactive effort of 15 tons at the rate of 2 tons per min. During compaction the temperature of the mix was not allowed to fall below 145 C.

## Results of Compaction

Tables 1, 2, and 3 give examples of the analyses conducted to study the orientation pattern of the specimens prepared by field roller, laboratory roller, and static method of compaction. The following computations complete the analyses.

Compacted by Field Roller

$$C = \frac{\sum X \cos\theta}{\sqrt{\sum \sin^2\theta}} = 0.95$$

$$C^2 = 0.90$$

$$S = \frac{\sum X \sin\theta}{\sqrt{\sum \sin^2\theta}} = 3.71$$

$$S^2 = 13.80$$

By testing $C^2 + S^2 = 14.70$ as $\chi^2$ with 2 degrees of freedom, it will be found that there is sufficient evidence against isotropicity. In other words, there is a preferred orientation and the mean orientation direction with respect to horizontal ($\theta_0$) will be

$$\tan\theta_1 = \frac{S}{C} = \frac{3.71}{0.95} = 3.91$$

$$\text{sign } \sin\theta_1 = \text{sign } S = \text{positve}$$

$$\theta_1 = 75.7 \text{ deg}$$

$$\theta_0 = 75.7 + (-80) = -4.3 \text{ deg}$$

Compacted by Laboratory Roller

$$C = \frac{\sum X \cos\theta}{\sqrt{\sum \cos^2\theta}} = \frac{7.94}{\sqrt{8.94}} = 2.66$$

$$C^2 = 7.06$$

$$S = \frac{\sin\theta}{\sqrt{\sum \sin^2\theta}} = \frac{12.89}{\sqrt{6.00}} = 5.25$$

$$S^2 = 27.55$$

$$C^2 + S^2 = 7.06 + 27.55 = 34.61$$

We now test 34.61 as $\chi^2$ with 2 degrees of freedom. This shows that the particles are oriented preferentially. The mean orientation direction ($\theta_0$) will be

$$\tan\theta_1 = \frac{S}{C} = \frac{5.25}{2.66} = 1.97$$

$$\text{sign } \sin\theta_1 = \text{sign } S = \text{positive}$$

$$\theta_1 = 63 \text{ deg}$$

$$\theta_0 = 63 + (-70) = -7 \text{ deg}$$

TABLE 1

SPECIMENS COMPACTED BY FIELD ROLLER

| Orientation Angle | Midpoint, θ | Observed Frequency, $O_i$ | Expected Frequency, $E_i$ | $O_i - E_i$ | $\chi = \frac{(O_i - E_i)}{\sqrt{E_i}}$ | cos θ | χ cos θ | sin θ | χ sin θ | $\cos^2 \theta$ | $\sin^2 \theta$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| -85 to -75 | -80 | 6 | 16 | -10 | -2.5 | -0.17 | +0.42 | -0.99 | +2.48 | 0.03 | 0.98 |
| -75 to -65 | -70 | 7 | 16 | -9 | -2.2 | +0.34 | -0.75 | -0.94 | +2.06 | 0.12 | 0.88 |
| -65 to -55 | -60 | 10 | 16 | -6 | -1.5 | -0.50 | +0.75 | -0.87 | +1.30 | 0.25 | 0.76 |
| -55 to -45 | -50 | 14 | 16 | -2 | -0.5 | -0.64 | +0.32 | -0.77 | +0.38 | 0.41 | 0.59 |
| -45 to -35 | -40 | 13 | 16 | -3 | -0.8 | -0.77 | +0.62 | -0.64 | +0.51 | 0.59 | 0.41 |
| -35 to -25 | -30 | 25 | 16 | +9 | +2.2 | -0.87 | -1.91 | -0.50 | -1.10 | 0.76 | 0.25 |
| -25 to -15 | -20 | 23 | 16 | +7 | +1.8 | -0.94 | -1.69 | -0.34 | -0.61 | 0.88 | 0.12 |
| -15 to -5 | -10 | 22 | 16 | +6 | +1.5 | -0.98 | -1.47 | -0.17 | -0.25 | 0.96 | 0.03 |
| -5 to +5 | 0 | 37 | 16 | +21 | +5.2 | +1.00 | +5.20 | 0.00 | 0.00 | 1.00 | 0.00 |
| +5 to +15 | +10 | 24 | 16 | +8 | +2.0 | +0.98 | +1.96 | -0.17 | -0.34 | 0.96 | 0.03 |
| +15 to +25 | +20 | 20 | 16 | +4 | +1.0 | +0.94 | +0.94 | -0.34 | -0.34 | 0.88 | 0.12 |
| +25 to +35 | +30 | 25 | 16 | +9 | +2.2 | +0.87 | +1.91 | -0.50 | -1.10 | 0.76 | 0.25 |
| +35 to +45 | +40 | 10 | 16 | -6 | -1.5 | +0.77 | -1.15 | -0.64 | +0.96 | 0.59 | 0.41 |
| +45 to +55 | +50 | 16 | 16 | 0 | 0 | +0.64 | 0.00 | -0.77 | 0.00 | 0.41 | 0.59 |
| +55 to +65 | +60 | 9 | 16 | -7 | -1.8 | +0.50 | -0.90 | -0.87 | +1.57 | 0.25 | 0.76 |
| +65 to +75 | +70 | 4 | 16 | -12 | -3.0 | +0.34 | -1.02 | -0.94 | +2.82 | 0.12 | 0.88 |
| +75 to +85 | +80 | 7 | 16 | -9 | -2.2 | +0.17 | -0.37 | -0.99 | +2.18 | 0.03 | 0.98 |
| Total | | 272 | | | | | +2.86 | | +10.52 | 9.00 | 8.04 |

TABLE 2

SPECIMENS COMPACTED BY LABORATORY ROLLER

| Orientation Angle | Midpoint, θ | Observed Frequency, $O_i$ | Expected Frequency, $E_i$ | $O_i - E_i$ | $\chi = \frac{(O_i - E_i)}{\sqrt{E_i}}$ | cos θ | χ cos θ | sin θ | χ sin θ | $\cos^2 \theta$ | $\sin^2 \theta$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| -75 to -65 | -70 | 4 | 15 | -11 | -2.84 | -0.34 | +0.96 | -0.94 | +2.67 | 0.12 | 0.88 |
| -65 to -55 | -60 | 3 | 15 | -12 | -3.10 | -0.50 | +1.55 | -0.87 | +2.70 | 0.25 | 0.76 |
| -55 to -45 | -50 | 8 | 15 | -7 | -1.81 | -0.64 | +1.16 | -0.77 | +1.39 | 0.41 | 0.59 |
| -45 to -35 | -40 | 11 | 15 | -4 | -1.03 | -0.77 | +0.79 | -0.64 | +0.66 | 0.59 | 0.41 |
| -35 to -25 | -30 | 14 | 15 | -1 | -0.26 | -0.87 | +0.23 | -0.50 | +0.13 | 0.76 | 0.25 |
| -25 to -15 | -20 | 26 | 15 | +11 | +2.84 | -0.94 | -2.67 | -0.34 | -0.97 | 0.88 | 0.12 |
| -15 to -5 | -10 | 21 | 15 | +6 | +1.55 | -0.98 | -1.52 | -0.17 | -0.26 | 0.96 | 0.03 |
| -5 to +5 | 0 | 47 | 15 | +32 | +8.26 | +1.00 | +8.26 | 0.00 | 0.00 | 1.00 | 0.00 |
| +5 to +15 | +10 | 28 | 15 | +13 | +3.36 | +0.98 | +3.29 | -0.17 | -0.59 | 0.96 | 0.03 |
| +15 to +25 | +20 | 18 | 15 | +3 | +0.77 | +0.94 | +0.72 | -0.34 | -0.26 | 0.88 | 0.12 |
| +25 to +35 | +30 | 15 | 15 | 0 | 0.00 | +0.87 | 0.00 | -0.50 | 0.00 | 0.76 | 0.25 |
| +35 to +45 | +40 | 7 | 15 | -8 | -2.06 | +0.77 | -1.58 | -0.64 | +1.32 | 0.59 | 0.41 |
| +45 to +55 | +50 | 9 | 15 | -6 | -1.55 | +0.64 | -0.99 | -0.77 | +1.19 | 0.41 | 0.51 |
| +55 to +65 | +60 | 5 | 15 | -10 | -2.58 | +0.50 | -1.29 | -0.87 | +2.24 | 0.25 | 0.76 |
| +65 to +75 | +70 | 4 | 15 | -11 | -2.84 | +0.34 | -0.97 | -0.94 | +2.67 | 0.12 | 0.88 |
| Total | | 220 | | | | | +7.94 | | +12.89 | 8.94 | 6.00 |

TABLE 3

SPECIMENS COMPACTED BY STATIC METHOD OF COMPACTION

| Orientation Angle | Midpoint, θ | Observed Frequency, $O_i$ | Expected Frequency, $E_i$ | $O_i - E_i$ | $\chi = \frac{(O_i - E_i)}{\sqrt{E_i}}$ | cos θ | χ cos θ | sin θ | χ sin θ | $\cos^2 \theta$ | $\sin^2 \theta$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| -65 to -55 | -60 | 4 | 16 | -12 | -3.00 | -0.50 | +1.50 | -0.87 | +2.61 | 0.25 | 0.76 |
| -55 to -45 | -50 | 5 | 16 | -11 | -2.75 | -0.64 | +1.76 | -0.77 | +2.12 | 0.41 | 0.59 |
| -45 to -35 | -40 | 9 | 16 | -7 | -1.75 | -0.77 | +1.35 | -0.64 | +1.12 | 0.59 | 0.41 |
| -35 to -25 | -30 | 20 | 16 | +4 | +1.00 | -0.87 | -0.87 | -0.50 | -0.50 | 0.76 | 0.25 |
| -25 to -15 | -20 | 18 | 16 | +2 | +0.50 | -0.47 | -0.47 | -0.34 | -0.17 | 0.88 | 0.12 |
| -15 to -5 | -10 | 20 | 16 | +4 | +1.00 | -0.98 | -0.98 | -0.17 | -0.17 | 0.96 | 0.03 |
| -5 to +5 | 0 | 34 | 16 | +18 | +4.50 | +1.00 | +4.50 | 0.00 | 0.00 | 1.00 | 0.00 |
| +5 to +15 | +10 | 25 | 16 | +9 | +2.25 | +0.98 | +2.20 | -0.17 | -0.38 | 0.96 | 0.03 |
| +15 to +25 | +20 | 25 | 16 | +9 | +2.25 | +0.94 | +2.11 | -0.34 | -0.77 | 0.88 | 0.12 |
| +25 to +35 | +30 | 36 | 16 | +20 | +5.00 | +0.87 | +4.35 | -0.50 | -2.50 | 0.76 | 0.25 |
| +35 to +45 | +40 | 14 | 16 | -2 | -0.50 | +0.77 | -0.38 | -0.64 | +0.32 | 0.59 | 0.41 |
| +45 to +55 | +50 | 6 | 16 | -10 | -2.50 | +0.64 | -1.60 | -0.77 | +1.92 | 0.41 | 0.59 |
| +55 to +65 | +60 | 9 | 16 | -7 | -1.75 | +0.50 | +0.87 | -0.87 | +1.52 | 0.25 | 0.76 |
| +65 to +75 | +70 | 4 | 16 | -12 | -3.00 | +0.34 | -1.02 | -0.94 | +2.82 | 0.12 | 0.88 |
| Total | | 229 | | | | | +11.58 | | +7.94 | 8.82 | 5.20 |

Compacted by Static Method of Compaction

$$C = \frac{\sum \chi \cos\theta}{\sqrt{\sum \cos^2\theta}} = \frac{11.58}{\sqrt{8.82}} = 3.90$$

$$C^2 = 15.20$$

$$S = \frac{\sum \chi \sin\theta}{\sqrt{\sum \sin^2\theta}} = \frac{7.94}{\sqrt{5.20}} = 3.48$$

$$S^2 = 12.20$$

$$C^2 + S^2 = 15.20 + 12.10 = 27.40$$

We now test 27.40 as $\chi^2$ with 2 degrees of freedom. This shows sufficient evidence against isotropicity (preferred orientation). The mean orientation direction ($\theta_0$) will be

$$\tan\theta_1 = \frac{S}{C} = \frac{3.48}{3.90} = 0.89$$

$$\text{sign } \sin\theta_1 = \text{sign } S = \text{positive}$$

$$\theta_1 = 40 \text{ deg}$$

$$\theta_0 = 40 + (-60) = -20 \text{ deg}$$

These results show that under the action of the field roller, for the particular mix employed, the stone particles are oriented preferentially with a mean orientation direction of about 4 deg with the horizontal. Moreover, it is clear that this orientation pattern may be more closely imitated by the special laboratory rolling compaction machine, for which the mean orientation direction was about 7 deg with the horizontal, than by the static method of compaction, for which the mean orientation direction was about 20 deg with the horizontal.

## CONCLUSIONS

As demonstrated, no method was available for the direct study of the orientation pattern of particles in bituminous materials, in spite of the fact that the need for it had been recognized by several researchers. A method has consequently been developed for particle orientation analysis in bound road materials, and it has been shown how the method can be extended and employed for unbound granular materials.

Also examined was the hypothesis that, under the action of the field roller, the particles will be oriented preferentially. It was found that the hypothesis is valid with, for the particular composition and compactive effort applied, a mean orientation direction of about 4 deg with the horizontal. This orientation pattern may be approximately imitated in the laboratory by a special roller compaction machine, giving a mean orientation direction of about 7 deg with the horizontal. Although the laboratory static method of compaction has been shown to produce preferential orientation of the particles, this is not as well developed as under the field roller, giving a mean orientation direction of about 20 deg with the horizontal.

## ACKNOWLEDGMENT

This work has been carried out in the Department of Transportation and Environmental Planning, University of Birmingham, England. Thanks are due to Professor J. Kolbuszewski, Head of the Department, for his interest and encouragement in all stages of this work.

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# Asphalt Content Determination By the Ignition Method

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A method for the rapid determination of asphalt content has been developed in which the weight of a sample before and after removal of the asphalt by burning is utilized. Virtually complete combustion of the asphalt is achieved by subjecting the asphaltic paving mixture sample to a very high temperature and an excess of oxygen in a special, but inexpensive, furnace using oxygen and butane as fuel.

The time required to complete a test varies with the sample size, the asphalt content of the mixture, and the temperature of the mixture before beginning the test. The time required for one operator to complete a test on a 1000-gm sample is approximately 25 minutes. The ignition test equipment can be used equally well in the field or the laboratory and is simple to operate.

Results indicate that the ignition method can be used to determine from single samples the asphalt content of hot-mix paving mixtures within ±½ percent and within ±¼ percent when the results from several samples are averaged. One aspect of the ignition method in its present conception is that preliminary calibration tests must be performed to assess correction factors for different operators and aggregate types.

•A NEW method for the rapid determination of the asphalt content of an asphaltic paving mixture has been developed that utilizes the weight of a sample of the mixture before and after removal of the asphalt by burning. This ignition method achieves virtually complete combustion of the asphalt by subjecting the paving mixture sample to a very high temperature and an excess of oxygen in a special, but inexpensive, furnace using butane as a fuel.

The ignition method, which has been undergoing continual development and evaluation over the past 2½ years, requires very little equipment and is suitable for both field and laboratory use. In addition, it does not require the constant attention of an operator while the test is in progress.

## EQUIPMENT

The equipment and materials required for the ignition method consist of (a) a special furnace; (b) a supply of welding grade oxygen; (c) a supply of butane, propane, or similar gas; (d) a high-capacity balance sensitive to ± 0.1 gm; (e) a device to ignite the butane-oxygen mixture; and (f) a pair of thick asbestos gloves. Figure 1 shows most of the equipment in position for conducting a test.

The furnace consists of three units: the lower unit or weighing pan, the middle unit or combustion chamber, and the upper unit or fines collector.

Paper sponsored by the Committee on Construction Practices—Flexible Pavement and presented at the 48th Annual Meeting.

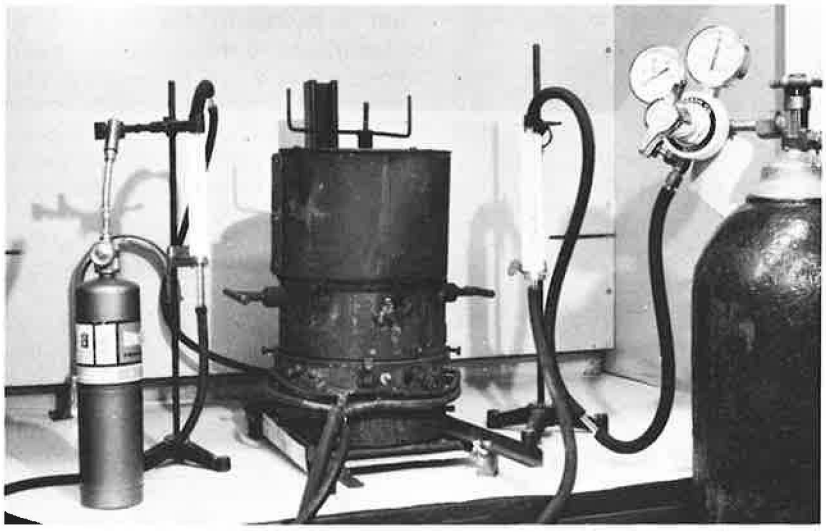

Figure 1. Ignition method equipment in position for a test.

The lower unit (Fig. 2) is a steel pan that supports the sample basket and catches any material passing through the sample basket. The sample basket consists of a Monel rod frame, a No. 40 mesh Inconel wire cloth sidewall, a No. 4 mesh Inconel wire cloth bottom, and a No. 40 mesh Inconel wire cloth sub-bottom. The sub-bottom is slightly dished to insure that material passing through the No. 4 mesh bottom will pass through the hottest zone in the combustion chamber on its way to the weighing pan.


Figure 2. Weighing pan and sample basket assembly: (a) disassembled sample basket and weighing pan with carrying frame, and (b) weighing pan and sample basket in weighing position.

The middle unit (Fig. 3a) consists of two pieces of steel tubing. The lower piece supports two separate manifolds, one supplying butane to 4 nozzles and one supplying oxygen to 4 nozzles. Also in this lower piece is an opening to permit ignition of the butane and oxygen mixture. The upper piece of the combustion chamber serves as a spacer to keep the fines collector clear of the sample basket. This upper piece has an opening that permits viewing of the sample while a test is in progress to determine when combustion of the asphalt is complete. Attached to this upper piece is a trolley unit that fits inside the vertical box member, thus enabling the combustion chamber to be raised and lowered without difficulty.

Associated with the combustion chamber are flow gages (Fig. 3b) used to regulate the amounts of oxygen and butane for each test. Sensitive flow gages were necessary because the extremely small movement of the hand-operated gas valves could

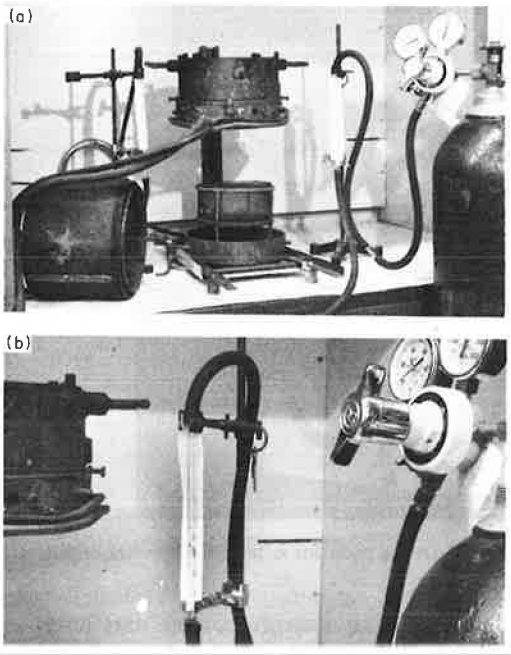


Figure 3. (a) Disassembled burner, and (b) oxygen flow gage.

not be duplicated for each test without the assistance of these gages. Each flow gage consists of a length of glass tubing in which a lightweight bead is suspended. The bead in the oxygen flow gage is attached to a piece of elastic thread and the bead in the butane flow gage is attached to a hairspring.

The upper unit consists of several pieces whose collective function is to cool the exhaust gases, trap carbon particles carried in the exhaust gases, and return aggregate fines to the combustion chamber. Figure 4 shows the fines collector in two stages of assembly, and the fully assembled fines collector appears in the lower left-hand portaion of Figure 3a. Included in the fines collector is a combination chopper-stirrer that is used to keep the burning mixture loose in the sample basket. It is estimated that the furnace could be custom made for less than $250.

## PROCEDURE

The weighing pan and sample basket assembly are weighed empty before introducing a sample of the paving mixture. After the weight of the pan-basket-sample combination is recorded, the assembly is placed in position under the combustion chamber. The chamber is lowered until it is firmly seated on the weighing pan, and then the fines collector is secured in position directly above the combustion chamber.

After the furnace is assembled, oxygen is allowed to enter the combustion chamber, and the furnace ignitor is lit and its flame is directed through the lower opening in the chamber. Only then is the butane allowed to enter the combustion chamber. (Strict adherence to this sequence will avoid explosions due to an accumulation of unburned butane in the combustion chamber.) The butane and oxygen valves are then adjusted

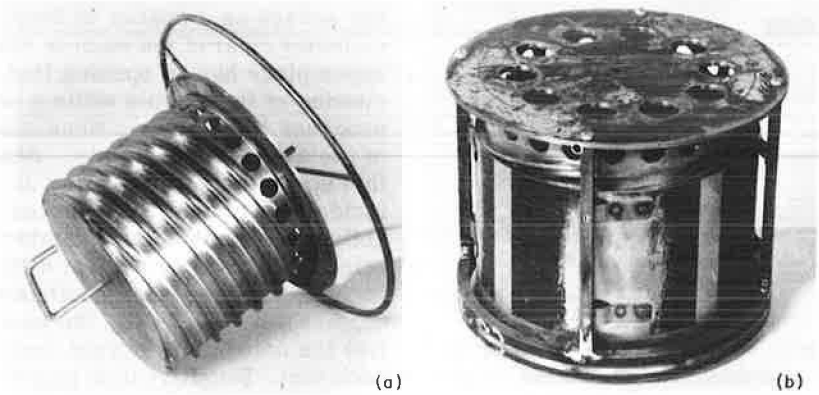


Figure 4. Fines collector: (a) innermost assembly and early version of stirrer, and (b) inner cover and frame for outer cover in place.

until the flow gages indicate that the desired amounts of fuel are entering the combustion chamber.

When the sample ignites, it is periodically agitated by the combination stirrer-chopper until the flames die out. At this point the oxygen and butane flows are terminated, the fines collector is removed, the combustion chamber is raised, and the pan-basket assembly is removed to the balance with the aid of the carrying frame. A weighing is made immediately, the burned mixture is quickly removed, and the empty pan-basket assembly is weighed again.

The two weighings made prior to the burning of the sample establish the weight of the mixture and the two weighings taken after the burning establish the weight of the mixture residue. The difference between these two weights is the weight loss due to burning, and it is equal to the weight of asphalt removed by the burning and a small loss in weight of the aggregate due to the high temperature (approximately 1500 F) in the combustion chamber. The correction for the aggregate weight loss must be determined by burning a mixture with a known asphalt content.

The time needed to complete a test varies with the asphalt content of the mixture, the temperature of the mixture prior to starting the test, and the size of sample. Mixtures with high asphalt contents and large sample sizes take longer to burn; preheated mixtures take less time. The asphalt content of mixtures at room temperature and with normal asphalt contents can be determined in 20 to 35 minutes with 10 to 25 minutes of this being the actual burning time for a 1000-gm sample.

Tests conducted during the development and evaluation of the ignition method used approximately 20 cu ft of oxygen and less than 1 cu ft of butane per test. It is estimated that for 1000-gm samples and a normal range of asphalt contents, the combined oxygen and butane cost would be approximately 20 cents per test.

## TEST METHOD EVALUATION

Evaluation of the ignition method included consideration of (a) time per test, (b) aggregate weight loss, (c) aggregate type, (d) aggregate breakage, and (e) test reproducibility.

### Time Per Test

During the development of the ignition method it became apparent that sample size affected the burning time, and burning times varied for a given sample size. A program to evaluate the variation in burning times was carried out by having one operator perform a number of tests on one mixture. Table 1 gives the total time needed to complete each test. The time required for completely burning a 1000-gm sample was 30 minutes or less, and several operators consistently completed 8 or more tests in a 4-hr period. Table 1 indicates that the length of time required to complete a test increases with the sample size, but the test time per gram of sample decreases with increasing sample size.

TABLE 1

RELATIONSHIP OF SAMPLE SIZE AND TIME PER TEST

| Sample No. | Total Time for Test[a] (minutes) | | | |
|---|---|---|---|---|
| | Mix 7[b] | Mix 8[b] | Mix 9[b] | Mix 10[b] |
| 1 | 21.5 | 23.2 | 23.5 | 29.0 |
| 2 | 15.6 | 22.3 | 22.5 | 31.0 |
| 3 | 13.0 | 20.5 | 21.3 | 30.5 |
| 4 | 13.0 | 21.5 | 20.2 | 32.2 |
| 5 | 13.5 | 20.2 | 19.5 | 27.8 |
| 6 | 15.8 | 19.5 | 19.0 | 29.3 |
| 7 | 13.5 | 19.0 | 21.8 | 29.5 |
| 8 | 13.5 | 20.7 | 23.8 | 28.8 |
| 9 | 15.3 | 19.8 | 22.5 | 28.8 |
| 10 | 13.7 | 19.5 | 22.2 | 29.0 |
| Average time | 14.8 | 20.6 | 21.6 | 29.6 |
| Sample size | 500 gm | 750 gm | 1000 gm | 1500 gm |

[a]Time measured from start of test to start of next test.
[b]Mixture is identified and described in Tables 5 and 6.

### Aggregate Weight Loss

Early work indicated that heating aggregates alone in the burner did not subject the aggregate to the same conditions that prevail when burning asphalt is present. Therefore, two

series of burnings were conducted on asphalt-aggregate samples prepared with graded crushed stone aggregate. One series consisted of samples prepared in the basket by pouring on a predetermined weight of asphalt. The second series consisted of samples that were mixed one at a time in a mechanical mixer.

The difference between weight loss on burning and known weight of the asphalt was assumed to be aggregate weight loss since the exact amounts of asphalt and aggregate in the sample were known. The weight discrepancy in the second series was also assumed to be aggregate weight loss; however, in this case it was necessary to calculate the amounts of asphalt and aggregate in the sample by correcting the batch weights by the amounts of asphalt and aggregate retained in the mixing bowl after discharging the mix into the sample basket.

In neither case was there an obvious relationship between asphalt content and aggregate weight loss. The variability of these results is due, in part, to laboratory technique because several different laboratory technicians conducted the tests. There is, however, a distinct difference in the magnitude of aggregate weight loss when the unmixed samples are compared with the mixed samples. The average aggregate weight loss for 21 unmixed samples was 4.4 gm, while the average aggregate weight loss for 17 mixed samples was 7.1 gm. This difference supports earlier observations that burning asphalt produces a more severe heating condition. The unmixed samples did not have a uniform coating of asphalt around every aggregate particle, and consequently the aggregate particles were not heated to temperature extremes responsible for aggregate breakdown.

Results of several hundred tests conducted on plant-mixed samples confirm that there is an aggregate weight loss. Average apparent losses are tabulated in the last column of Table 7. They were obtained by taking the difference between the average asphalt content indicated by the ignition method and the job mix asphalt content. The exact value of the aggregate contribution cannot be determined from the values listed in Table 7 because several aggregate types are represented. In all likelihood, the asphalt content as determined by the centrifuge test is not the true asphalt content of the mixture (1, 2), and hence it was not used to calculate the apparent aggregate contribution. Use of the asphalt content as calculated from actual batch weights would have been preferred for determining the apparent aggregate contribution, but batch weights were not available for any of the mixes.

## Aggregate Type

Burnings were made to determine if various aggregate types would lose more weight than others when subjected to the high temperatures present in the combustion chamber. Much of the data collected did not indicate significant differences in aggregate weight loss.

Results for one series of tests in which a single operator conducted all the tests are given in Table 2. Two mixes were made in the laboratory and each contained the same amount of asphalt, fine aggregate, and coarse aggregate; however, limestone was used as the coarse aggregate in one mix and granite gneiss was used in the other mix. The results of this series of tests indicated that there was a greater weight loss for mixes made with limestone than for mixes made with granite gneiss.

TABLE 2

INFLUENCE OF AGGREGATE TYPE ON INDICATED ASPHALT CONTENT

| Coarse Aggregate Type | Size of Mix[a] (gm) | Total Weight Loss per 1000 Grams of Mix (gm) | Indicated Asphalt Content (percent) | Estimated Asphalt Content of Mix[b] (percent) |
|---|---|---|---|---|
| Granite gneiss | 3493 | 54.2 | 5.4 | 5.4 |
| Limestone | 3505 | 59.0 | 5.9 | 5.4 |

[a]Mix split into three samples prior to burning.
[b]Batched asphalt content corrected by subtracting asphalt retained on mixing equipment.

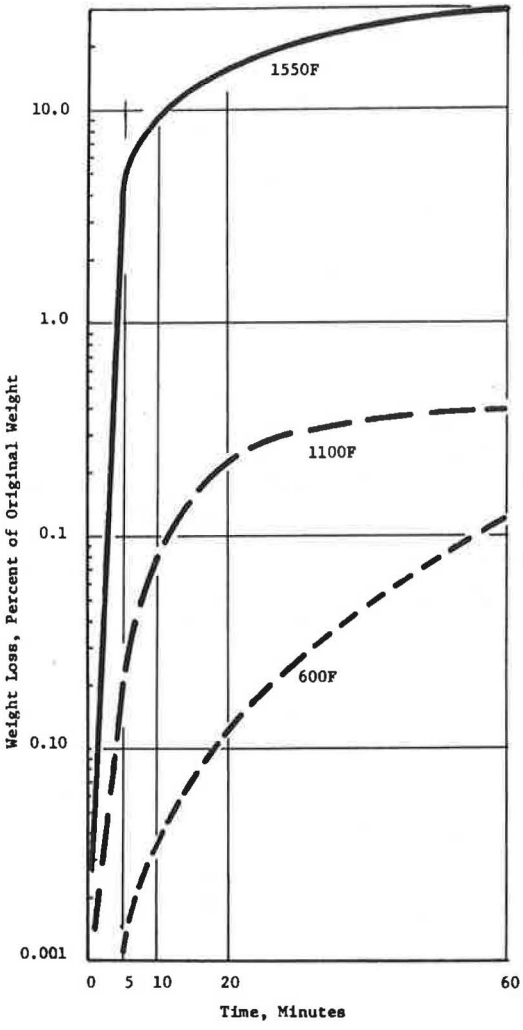


Figure 5. Weight loss—time relationships for limestone at 600, 1100, and 1550 F.

X-ray diffraction tests on granite gneiss were made before and after a normal ignition test. Test results indicated that with this aggregate no mineralogical changes took place during burning.

Additional laboratory testing was conducted to determine the weight loss that occurred at elevated temperatures for both limestone and granite gneiss. Oven-dry samples of these aggregates were placed in an electric furnace and heated for 1 hour. Test temperatures were 600, 1100, and 1550 F. Periodically, the samples were removed from the furnace and weighed on an analytical balance.

The weight loss for granite gneiss was negligible for all test temperatures used in this study. Results from the weight loss on heating test for limestone are shown in Figure 5. Although the weight loss at 600 F is negligible, the weight loss for limestone becomes significant for higher test temperatures and after longer periods of heating. The limestone lost approximately 30 percent of its oven-dry weight after being heated to 1550 F for 1 hour.

After heating, the limestone was frangible and exhibited a distinct color change. Although frangibility and color changes in limestone were not evident after normal 25-minute ignition tests, results of the loss on heating tests performed at 1550 F indicate that precaution must be exercised in using the ignition method if it involves prolonged heating of aggregates that decompose significantly at elevated temperatures.

The weight losses occurring at elevated temperatures in the electric furnace were also corrobrated by results of burnings conducted on mixes 22 and 23 (Table 7). These two mixes were made with a limestone aggregate and appeared to have a greater "apparent aggregate contribution" than mixes 1 through 10, which were made with granite gneiss.

Mixes 11 through 25 (Table 7) were purposely selected to give, among other things, a variety of aggregate sources. The results obtained were not adequate for a complete statistical analysis, but the existence of several possible interactions between effects, such as mix type, aggregate type, and operator, is likely.

## Aggregate Breakage

The possibility of aggregate breakage due to temperature-induced stresses was investigated by determining the aggregate gradation of several 1000-gm samples before and after burning.

The laboratory phase of this part of the investigation involved three separate aggregate conditions. In the first condition, the aggregate was batched as if a mixture were to be made and then a sieve analysis was conducted. In the second condition, a sieve analysis was conducted on aggregate that was batched, mixed with asphalt in the mechanical mixer, and then washed with a solvent to remove the asphalt from the mixture.

TABLE 3

SIEVE ANALYSIS RESULTS: STANDARD LABORATORY MIX[a]

| Sieve Size | Percent Passing | | | | | |
|---|---|---|---|---|---|---|
| | After Batching | After Mixing and Solvent Extraction (2 samples) | | After Burning (3 samples) | | |
| ½ in. | 100 | 100 | 100 | 100 | 100 | 100 |
| No. 4 | 54.4 | 52.8 | 53.5 | 63.2 | 61.6 | 61.1 |
| No. 8 | 39.6 | 36.0 | 38.2 | 46.2 | 45.7 | 44.5 |
| No. 30 | 18.6 | 17.6 | 17.3 | 22.0 | 22.3 | 22.3 |
| No. 200 | 3.5 | 3.5 | 3.1 | 3.6 | 3.5 | 4.6 |

[a]All aggregate from the Liberty, South Carolina, quarry of Campbell Limestone Company.

In the third condition, a sieve analysis was conducted on a mixture after removal of the asphalt by burning. In each case, samples containing 6 percent asphalt by weight of the aggregate were prepared from granite gneiss aggregate of the same type, amount, and gradation. The results of these tests are given in Table 3. It is evident from these results that the larger sizes were degraded more than the smaller sizes.

Several plant-mixed samples of approximately 1000 gm were also burned, and sieve analyses were conducted on the aggregate residues. Results of this series of tests, given in Table 4, also indicate that the larger aggregate sizes were subject to some degradation.

## Test Reproducibility

Over a period of approximately one year, 22 samples of plant mixtures were obtained for the primary purpose of checking the reproducibility of results obtained by the ignition method.

Data for 262 burnings conducted on the samples of plant mixtures are given in Table 5. Table 6 identifies each mixture and Table 7 summarizes various properties of each mixture.

Information summarized in Tables 5 and 7 indicates that a series of burnings on the same mixture gives total weight losses that do not fluctuate widely. The standard deviation for each group of burnings is small, except for mixtures 17 and 22, and the coefficient of variation is less than 5 percent for 19 of the 25 mixtures listed. An analysis

TABLE 4

SIEVE ANALYSIS RESULTS: PLANT MIXTURES

| Sieve Size | Percent Passing | | | | |
|---|---|---|---|---|---|
| | Surface Mix[a] | | Binder Mix[a] | | |
| | Burned Sample | Job-Mix Formula | Burned Sample | Job-Mix Formula | S. C. State Highway Dept. Analysis |
| 1 in. | | | 100 | 100 | 100 |
| ¾ in. | | | — | 93 | 98.5 |
| ½ in. | 100 | 100 | 82.6 | 75 | 75.5 |
| ⅜ in. | — | 90 | — | — | — |
| No. 4 | 71.9 | 68 | 40.5 | 36 | 35 |
| No. 8 | 60.7 | 56 | 33.1 | — | — |
| No. 10 | — | — | — | 26 | 28.5 |
| No. 30 | 37.8 | 33 | 22.5 | — | — |
| No. 100 | — | 14 | — | — | — |
| No. 200 | 8.4 | 7 | 3.8 | — | — |

[a]Mixed at the Liberty, South Carolina, plant of Sloan Construction Company.

TABLE 5

DATA FOR BURNINGS MADE ON PLANT MIXTURES

| Sample No. | Total Weight Loss Due to Burning (gm) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Mix 6 | Mix 7 | Mix 8 | Mix 9 | Mix 10 | Mix 11 | Mix 12 | Mix 13 |
| 1 | 63.0 | 59.1 | 58.6 | 66.0 | 55.4 | 36.7 | 30.9[a] | 49.8 | 63.4 | 99.0[a] | 66.0 | 60.0 | 52.0 |
| 2 | 64.3 | 57.6 | 58.9 | 63.6 | 52.6 | 36.7 | 30.9 | 48.0 | 63.7 | 97.3 | 67.0 | 59.8 | 54.0 |
| 3 | 68.9 | 56.5 | 59.5 | 65.7 | 53.1 | 35.8 | 31.1 | 50.3 | 63.0[a] | 96.0 | 65.6 | 60.0 | 51.5 |
| 4 | 65.3 | 57.5 | 61.0 | 61.4 | 53.3 | 30.7 | 31.8 | 50.6 | 64.2 | 91.7[a] | 66.0 | 63.0 | 49.0 |
| 5 | 65.3 | 55.8 | 62.0 | 63.0 | 55.8 | 38.2 | 31.8 | 46.0[a] | 65.1 | 93.6 | 65.0 | 60.2 | 55.0 |
| 6 | 67.5 | 61.6 | 64.0 | 61.1 | 55.0 | 41.3 | 32.6[a] | 47.7 | 63.7 | 96.0 | 67.5 | 60.5 | 55.0 |
| 7 | 62.3 | 63.3 | 62.5 | 67.9 | 56.0 | 37.1 | 32.9 | 47.9 | 62.9[a] | 91.3[a] | 66.2 | 58.0 | 56.0 |
| 8 | 63.5 | 61.2 | 63.8 | 68.0 | 52.0 | 38.0 | 32.3 | 46.2[a] | 66.0 | 95.5 | 67.0 | 56.0 | 50.8 |
| 9 | 62.8 | 59.3 | 62.1 | 69.3 | 56.0 | 37.9 | 30.8 | 46.8 | 63.2 | 96.1 | 65.0 | 56.5 | 54.0 |
| 10 | 63.0 | 59.8 | 62.3 | 63.7 | 54.0 | 38.1 | 32.9 | 49.3 | 66.3 | 96.1 | 63.6[b] | 54.4 | 55.5 |
| 11 | 64.9 | 58.5 | 62.9 | 69.9 | 50.6 | | | | | | 66.0 | 53.0 | 51.1 |
| 12 | 62.1 | 59.0 | 63.2 | 68.8 | 61.7 | | | | | | | | |
| 13 | 67.0 | 59.8 | 58.3 | 68.3 | 57.6 | | | | | | | | |
| 14 | | 56.1 | 59.5 | | 52.8 | | | | | | | | |
| 15 | | | | | 53.9 | | | | | | | | |
| Sample size | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 500 gm | 750 gm | 1000 gm | 1500 gm | 1000 gm | 1000 gm | 1000 gm |
| Operator | A | B | C | A | A | B | D | D | D | D | E | E | E |

| Sample No. | Total Weight Loss Due to Burning (gm) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Mix 14 | Mix 15 | Mix 16 | Mix 17 | Mix 18 | Mix 19 | Mix 20 | Mix 21 | Mix 22 | Mix 23 | Mix 24 | Mix 25 |
| 1 | 64.7 | 70.7 | 73.3 | 42.5 | 42.4 | 54.1 | 67.6 | 54.4 | 48.9 | 88.9[b] | 71.4 | 66.7 |
| 2 | 68.0[b] | 71.8 | 68.1 | 40.4 | 51.2 | 51.3[b] | 66.4 | 52.5 | 58.6 | 88.6[b] | 70.0 | 70.2 |
| 3 | 68.6 | 66.7[b] | 77.8 | 36.8 | 44.4 | 57.3 | 70.8 | 53.0 | 63.6[b] | 96.8 | 66.7 | 67.6 |
| 4 | 65.0[b] | 70.1 | 78.0[b] | 41.3 | 47.8 | 62.8 | 66.3 | 53.6 | 53.6 | 94.8 | 66.4 | 66.5 |
| 5 | 63.9[b] | 68.5 | 74.1[b] | 49.7[b] | 44.3 | 53.1 | 65.0 | 54.7 | 61.5[b] | 89.7[b] | 72.1 | 66.9 |
| 6 | 67.0 | 67.9 | 73.4[b] | 42.7 | 48.8 | 60.5 | 68.7[b] | 53.0 | 47.9 | 90.5 | 69.7 | 65.8 |
| 7 | | 72.1 | 74.7 | 48.7[b] | 47.3 | 60.2 | 65.6 | 53.0[b] | 32.7 | 92.0 | 69.8 | 67.5 |
| 8 | | 70.6 | 73.6 | 56.5 | 44.7[b] | 54.3 | 64.6 | 53.7 | 63.4[b] | 93.1 | 67.6 | 66.5 |
| 9 | | 68.5[b] | 76.2 | 55.2 | 46.8 | 59.2 | 65.7 | 54.0 | | | 65.6 | |
| 10 | | 68.4 | 72.4 | 47.7 | 49.8 | 54.6[b] | 67.2 | 54.1 | | | 68.9 | |
| Sample size | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm | 1000 gm |
| Operator | E | F | F | F | F | F | F | F | F | F | F | F |

[a]Sample was first sample to be burned on a given day.

[b]Appearance of sample after burning indicated complete combustion was not obtained.

TABLE 6

IDENTIFICATION OF PLANT MIXTURES IN TABLE 5

| Mix No. | Mixture Type[a] | Mixture Source | Aggregate Designation | Aggregate Type |
|---|---|---|---|---|
| 1, 2, 3 | Surface 1 | Liberty, S.C., Sloan Const. Co. | Liberty | Granite gneiss |
| 4 | Surface 1 | Liberty, S.C., Sloan Const. Co. | Liberty | Granite gneiss |
| 5 | Binder 2 | Liberty, S.C., Sloan Const. Co. | Liberty | Granite gneiss |
| 6 | Black base | Liberty, S.C., Sloan Const. Co. | Liberty | Granite gneiss |
| 7, 8, 9, 10 | Surface 1 | Liberty, S.C., Sloan Const. Co. | Liberty | Granite gneiss |
| 11 | Surface 1 | Columbia, S.C., Sloan Const. Co. | Palmetto | Granite |
| 12 | Binder 2 | Greenville, S.C., Ashmore Bros., Inc. | Lakeside | Granite gneiss |
| 13 | Binder 2 | Pacolet, S.C., Sloan Const. Co. | Pacolet | Granite gneiss |
| 14, 15 | Surface 1 | Columbia, S.C., Sloan Const. Co. | Palmetto | Granite |
| 16 | Surface 1 | Greenwood, S.C., Sloan Const. Co. | Palmetto | Granite |
| 17 | Black base | Columbia, S.C., Sloan Const. Co. | Palmetto | Granite |
| 18 | Black base | Pacolet, S.C., Sloan Const. Co. | Pacolet | Granite gneiss |
| 19 | Binder 2 | Columbia, S.C., Sloan Const. Co. | Palmetto | Granite |
| 20 | Surface 1 | Pacolet, S.C., Sloan Const. Co. | Pacolet | Granite gneiss |
| 21 | Sand asphalt | Kershaw, S.C., Asphalt Products Corp. | Kershaw | Quartz and feldspar |
| 22 | Black base | Charlotte, N.C., Rea Const. Co. | Kings Mtn. | Limestone |
| 23 | Surface 1 | Charlotte, N.C., Rea Const. Co. | Kings Mtn. | Limestone |
| 24 | Surface 1 | Greenville, S.C., Ashmore Bros., Inc. | Lakeside | Granite gneiss |
| 25 | Surface 1 | Sumter, S.C., Asphalt Products Corp. | Hagood | Quartz and feldspar |

[a]Each mixture manufactured to conform with applicable Highway Department specifications.

TABLE 7

ANALYSIS OF BURNINGS MADE ON PLANT MIXTURES

| Mix No. | Average Total Weight Loss (gm) | Range | n[a] | s[b] | C[c] (percent) | Indicated Asphalt Content (percent by wt. of mix) (A) | Asphalt Content by Centrifuge Test (percent) | Job Mix Asphalt Content (percent) (B) | Apparent Aggregate Contribution (percent) (A) - (B) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 64.6 | 62.1-68.9 | 13 | 2.12 | 3.3 | 6.46 | NA[d] | 6.0 | +0.5 |
| 2 | 58.9 | 55.8-63.2 | 14 | 2.16 | 3.6 | 5.89 | NA | 6.0 | -0.1 |
| 3 | 61.3 | 58.3-64.0 | 14 | 2.01 | 3.3 | 6.13 | NA | 6.0 | +0.1 |
| 4 | 65.9 | 61.1-69.9 | 13 | 3.05 | 4.6 | 6.59 | 6.1 | 6.1 | +0.5 |
| 5 | 54.7 | 50.6-61.7 | 15 | 2.68 | 4.9 | 5.47 | 4.6 | 4.6 | +0.9 |
| 6 | 37.0 | 30.7-41.3 | 10 | 2.67 | 7.2 | 3.70 | NA | 3.9 | -0.2 |
| 7 | 31.8 | 30.8-32.9 | 10 | 0.8 | 2.5 | 6.36 | 6.1 | 6.0 | +0.4 |
| 8 | 48.3 | 46.0-50.6 | 10 | 1.7 | 3.5 | 6.44 | 6.1 | 6.0 | +0.4 |
| 9 | 64.2 | 62.9-66.3 | 10 | 1.2 | 1.9 | 6.42 | 6.1 | 6.0 | +0.4 |
| 10 | 95.3 | 91.3-99.0 | 10 | 2.4 | 2.5 | 6.35 | 6.1 | 6.0 | +0.4 |
| 11 | 65.9 | 63.6-67.5 | 11 | 1.1 | 1.7 | 6.59 | 6.0 | 6.1 | +0.5 |
| 12 | 58.3 | 53.0-63.0 | 11 | 3.0 | 5.1 | 5.83 | 4.8 | 4.8 | +1.0 |
| 13 | 53.1 | 49.0-56.0 | 11 | 2.3 | 4.3 | 5.31 | 4.5 | 4.7 | +0.6 |
| 14 | 66.2 | 63.9-68.6 | 6 | 1.9 | 2.9 | 6.62 | 6.0 | 6.1 | +0.5 |
| 15 | 69.5 | 66.7-72.1 | 10 | 1.8 | 2.7 | 6.95 | 6.0 | 6.1 | +0.9 |
| 16 | 74.2 | 68.1-78.0 | 10 | 2.9 | 3.9 | 7.42 | NA | NA | — |
| 17 | 46.2 | 36.8-56.5 | 10 | 6.5 | 14.1 | 4.62 | 4.0 | NA | — |
| 18 | 46.8 | 42.4-51.2 | 10 | 2.8 | 6.0 | 4.68 | 4.0 | NA | — |
| 19 | 56.7 | 51.3-62.8 | 10 | 3.8 | 6.7 | 5.67 | 4.8 | 5.0 | +0.7 |
| 20 | 66.8 | 65.0-70.8 | 10 | 1.9 | 2.8 | 6.68 | 5.9 | 6.2 | +0.5 |
| 21 | 53.6 | 52.5-54.7 | 10 | 0.7 | 1.3 | 5.36 | NA | 4.5 | +0.9 |
| 22 | 53.8 | 32.7-63.6 | 8 | 10.5 | 19.5 | 5.38 | NA | 4.0 | +1.4 |
| 23 | 91.8 | 88.6-96.8 | 8 | 2.9 | 3.2 | 9.18 | NA | 6.0 | +3.2 |
| 24 | 68.8 | 65.6-72.1 | 10 | 2.2 | 3.2 | 6.88 | 6.2 | 6.0 | +0.9 |
| 25 | 67.2 | 65.8-70.2 | 8 | 1.3 | 1.9 | 6.72 | 5.9 | 5.5 | +1.2 |

[a]Number of burnings.
[b]Standard deviation (total weight loss).
[c]Coefficient of variation.
[d]Value not available.

TABLE 8

ASPHALT CONTENT BY IGNITION METHOD AND CENTRIFUGE TEST

| Mix Identification |
|---|
| Mix No.: 26<br>Mixture type: Surface 1<br>Mixture source: Liberty, S.C., Sloan Construction Co.<br>Aggregate designation: Liberty<br>Aggregate type: Granite gneiss<br>Job-mix asphalt content: 6.0 percent<br>Centrifuge test asphalt content: 6.02 percent (test run at batch plant)<br>Sample size: 1000 gm |

Test Results

| Item | Indicated Asphalt Content (percent by wt. of mix) | |
|---|---|---|
| | Ignition Method | Centrifuge Test[a] |
| Sample 1 | 6.45 | 6.23 |
| Sample 2 | 6.74 | 6.09 |
| Sample 3 | 6.50 | 5.86 |
| Sample 4 | 6.74 | 5.95 |
| Sample 5 | 6.41 | 5.93 |
| Sample 6 | 6.67 | 6.00 |
| Sample 7 | 6.49 | 6.14 |
| Sample 8 | 6.50 | 6.05 |
| Sample 9 | 6.30 | 5.53 |
| No. of samples | 9 | 9 |
| Avg. indicated A/C | 6.53% | 5.98% |
| Range of indicated A/C | 0.44 | 0.70 |
| Standard deviation | 0.14 | 0.20 |
| Coefficient of variation | 2.2% | 3.3% |

[a]Tests performed by South Carolina State Highway Department personnel.

of the ranges of total weight loss values for the mixtures indicated that, for most of the mixtures listed, the range for indicated asphalt contents of a particular mixture would be 1 percent or less. It was noted from data obtained for mixtures 7, 8, 9, and 10 (Table 5) that data from the first burning of a series were apparently compatible with data from subsequent burnings even though the equipment was relatively cool at the beginning of the test. Weight-loss data tabulated for mixtures 11 through 13 in Table 5 are inconsistent because burning was not complete for a number of samples. Yet a close inspection of the data indicates that an incomplete burn did not result in a weight loss significantly different from weight losses for complete burns.

The presence of an operator effect is the only effect that could be established with some degree of confidence. This was done by performing a statistical analysis of the data listed for mixtures 1, 2, and 3, and mixtures 14 and 15. F-tests at an $\alpha$-level of 0.05 result in the acceptance of the hypothesis that the population variances are equal and t-tests at an $\alpha$-level of 0.05 result in the rejection of the hypothesis that the population means are equal. Hence, it can be said that the variances obtained are probably caused by the material and the test method, while the differences in the mean indicated asphalt contents probably can be attributed to the operators.

Limited testing was done to determine if the reproducibility of results obtained by the ignition method is any better than the reproducibility of results obtained by using the centrifuge test. Table 8 gives the asphalt contents obtained by testing eighteen 1000-gm samples taken from a plant-made surface mix. The statistics given in Table 8 suggest that when the aggregate contribution is known, the ignition method is as good as the centrifuge test for determining the asphalt content of a surface mix.

## CONCLUSIONS

Tests on laboratory-made and plant-made mixtures have established that the ignition method can be used to determine from single samples the actual asphalt content of hot-mix asphaltic paving mixtures within ± ½ percent, and possibly within ± ¼ percent when the results from several samples are averaged and a calibration factor for the particular operator-mix combination is applied. The method may also be used for other types of mixtures provided that heating does not cause significant weight loss in the aggregates.

The time required for one operator to complete a test on a 1000-gm sample is approximately 30 minutes. The test method is simple and no more than one demonstration is needed to acquaint operators with proper use of the equipment. The equipment is not sophisticated and is estimated to cost approximately $250. The only significant operating cost is the cost of fuel, which was 20 cents per burn for 1000-gm samples. In spite of the fact that the ignition method has been evaluated on hot-mix asphaltic paving mixtures, it is not limited to this type of mixture.

## ACKNOWLEDGMENTS

The development and initial evaluation of the ignition method was done as part of NCHRP Project 10-4. Continuation of the evaluation program after expiration of the NCHRP contract was made possible by grants from the Faculty Basic Research Committee and the Department of Civil Engineering of Clemson University.

The authors are also grateful for the cooperation and assistance of members of the South Carolina Asphalt Pavement Association.

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# Merits of Adding Natural Rubber to Bituminous Road Surfacing Materials

P. D. THOMPSON, The Natural Rubber Producers' Research Association, London

During 15 years of cooperative research between the Natural Rubber Producers' Research Association and the United Kingdom Road Research Laboratory, investigations were undertaken to find the extent to which the properties of road bitumen and asphaltic materials are altered by the addition of natural rubber. The paper describes how the alterations in properties of road binders were defined initially with reference to standard tests used in the road industry. These showed that rubber addition improved the brittle properties, reduced the temperature susceptibility, and increased the softening point, and that the resultant binders were markedly elastic.

Concurrently with the laboratory research, an extended series of full-scale road experiments was undertaken to show how rubberized surfacings behaved under traffic in comparison with normal materials. The paper indicates the advantages from the addition of natural rubber, which vary with the type of surfacing laid. In surface-dressing or seal coats, rubber reduces fatting-up considerably and gives greater initial retention of chippings; in open-textured carpets, a marked increase in life was found. Both rolled asphalt and mastic asphalt are made much more resistant to cracking, and the stability of mixtures is increased by addition of natural rubber. It is also possible to increase the amount of binder by about one-eight (which in itself reduces cracking and disintegration) without impairing the stability of the mixtures.

•ALTHOUGH the study of rubberized asphalts originated in the last century, it is only in the last 40 years that serious consideration has been given to rubber-asphalt mixtures with any hope of widespread use. At this stage, part of the "rubber-for-roads" effort was engaged in developing solid rubber surfacings that, for reasons of cost and slipperiness, were not destined to be used to any great extent. Latex-cement surfacings were also investigated about the same time but, as with the earlier rubber-asphalt trials, development was slow and World War II stopped further effort.

Some work on the rheological properties of bitumen containing rubber was carried out in Holland and the United Kingdom, but most of the prewar work consisted of road trials. There are reports of road-layings in Holland, Singapore, the Dutch East Indies, the United Kingdom, and the United States. Nearly all were initiated by natural rubber interests when the price of rubber was exceptionally low. The trials established that the use of rubber as an additive had definite merit but there was little basic knowledge about why and in what circumstances it was most advantageous.

From 1947, further trials were started but it was clear that no progress would be made unless a more systematic and fundamental approach was adopted. Within a few years, investigations were started in various parts of the world, and in 1951 a joint program of both laboratory and full-scale research was started in England by the Road Research Laboratory and the British Rubber Producers' Research Association (later,

Paper sponsored by Committee on Characteristics of Bituminous Materials and presented at the 48th Annual Meeting.

the Natural Rubber Producers' Research Association). The present paper is concerned with the findings from this research.

In this work it was recognized that, while it was important to obtain fundamental data on the properties of rubber-asphalt mixtures and the factors influencing their variation, practical road experiments were also essential. Clearly the final acceptance of rubberized mixtures depends on the economic benefits actually accruing to the road engineer; full-scale road experiments were therefore started early in the program. It is convenient, however, to deal with the full-scale laboratory investigations separately.

## PROPERTIES OF ROAD BINDERS CONTAINING NATURAL RUBBER

A number of researchers have established that the major changes in physical properties of asphalt brought about by the addition of small quantities of natural rubber are in the viscous, elastic, and brittle properties (1). Also, the quantitative change in the properties of asphalts induced by rubber addition depends on the type of asphalt, form of rubber, blending procedure, and the history of the modified binder in terms of time and temperature of storage prior to testing.

### Effect of Rubber Form

Before the result of tests on the rheological properties of natural-rubber asphalts can be fully appreciated, it is necessary to understand how the form of rubber and the time and temperature of heating influence the results.

Natural rubber is blended into asphalt either in vulcanized or unvulcanized form and as a dry powder or latex. In practice, three forms are used: unvulcanized powder (containing inert filler), vulcanized powder, and unvulcanized latex (either evaporated or centrifuged).

The difference in effectiveness between unvulcanized powder or latex is dependent on the molecular weight of the rubber. This should be similar for both unvulcanized powder or latex. In practice, their long-term effects will be influenced by natural or added antioxidants present, and after blending, which involves a finite time at relatively high temperatures, differences in behavior are generally found to exist.

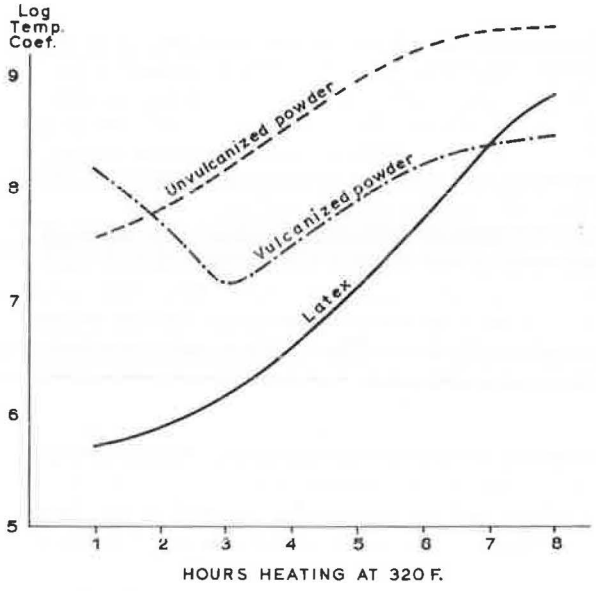


Figure 1. Change in temperature susceptibility with heating of rubber-asphalt blended with unvulcanized powder, vulcanized powder, and latex.

The difference between the properties of rubber-asphalt blended with vulcanized and unvulcanized rubber is considerably greater. Unvulcanized natural rubber dissolves readily in asphalt. In vulcanized rubber, the long-chain molecules are "welded" together into a network, and the rubber will not dissolve until the network is broken down by heating to shorter, branched chains, which are less effective.

The instrument used to measure the viscoelastic properties of the binders in the following investigations was a conicylindrical viscometer of 2 in. cylinder length, the diameters being 0.4 in. and 1.2 in. The annulus was 0.8 in. wide. A wide annulus was used to prevent interference with the results by fine particles of undissolved rubber. Full details have been described elsewhere (1).

Figures 1 and 2 show how the temperature susceptibility and change in softening point (ring and ball) of

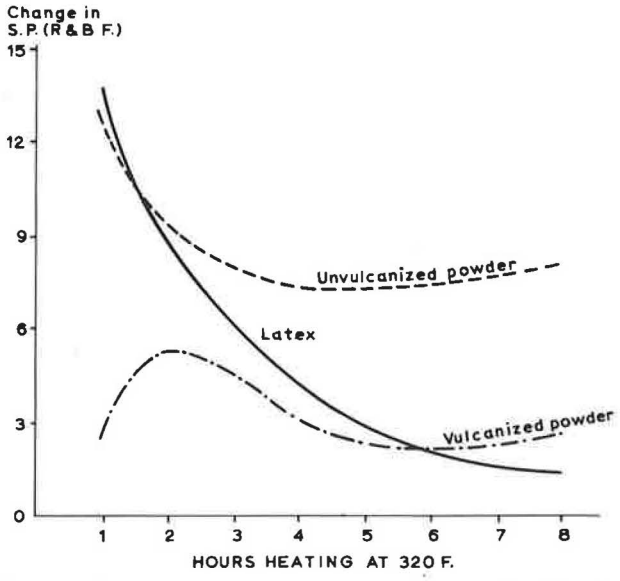


Figure 2. Change in softening point (ring and ball) with heating of rubber-asphalt blended with unvulcanized powder, vulcanized powder and latex.

rubber-bitumen blended from a Middle East bitumen alter during heating for an unvulcanized powder, a vulcanized powder, and latex. (The method of assessing temperature susceptibility is referred to later.) There is a continuous increase in effectiveness of the vulcanized rubber for the first few hours, during the time the rubber is broken down and dissolved. On prolonged heating more rubber is dissolved, but the continued breakdown of the initially dissolved rubber reduces its effectiveness.

It was recognized during the research that a single test method was desirable to assess the effectiveness of rubber in bitumen. It proved difficult, if not impossible, to assess the rubber content directly, so a method was evolved that measured the effectiveness of the rubber based on measurement of average molecular weight. The method has been published as part of the officially recommended specification for the United Kingdom (2). It depends on the recovery of the acetone-insoluble fraction of the binder on which are carried out specific viscosity and iodine value measurements. From the measurement of the specific viscosity of solution, the "effective rubber content" is calculated; the iodine value is a measure of the total unvulcanized dissolved rubber.

The effective rubber content is a measure of the useful rubber in the binder in terms of the amount of an undegraded standard natural rubber that would be required to give the same effect as the rubber actually present. It has been correlated with such properties as the change in softening point, elastic recovery, and temperature susceptibility. Figure 3 shows how the elastic recovery follows the effective rubber content. Measurements of elastic recovery were made using the conicylindrical viscometer. A torque of $10 \times 10^4$ dyne-cm was applied to the inner cylinder until the stress in the material at the surface of the inner cylinder was about 0.9. The torque was then suddenly removed and the elastic recovery was measured over a period of 15 min.

By using this method it has been possible to follow the effect

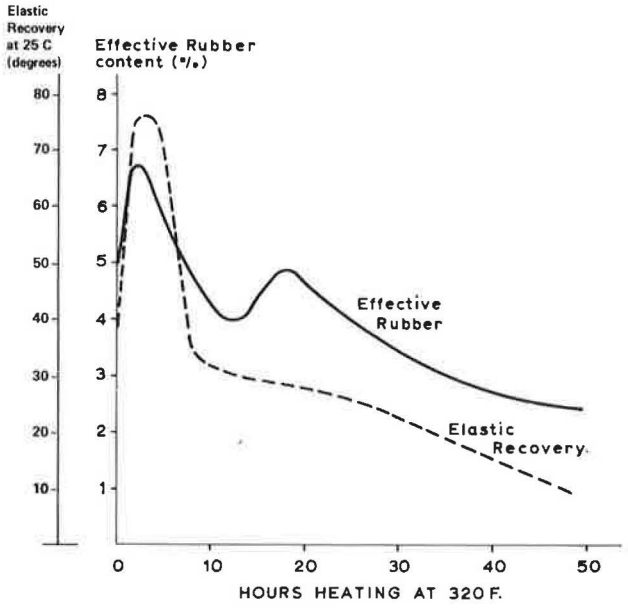


Figure 3. Relation between effective rubber content and elastic recovery during heating of rubber-asphalt.

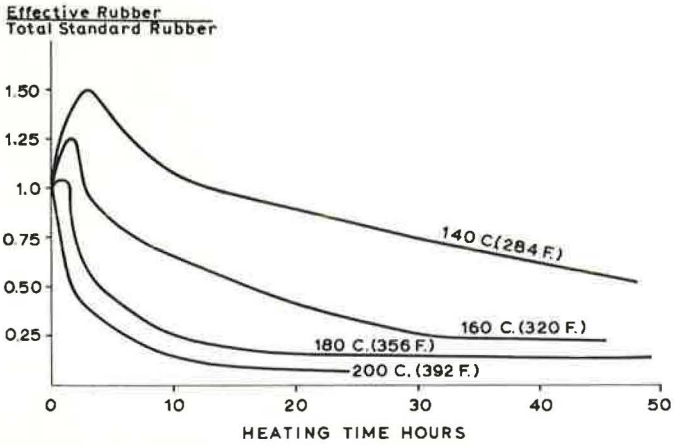


Figure 4. Variation of the effect of rubber in asphalt when heated at various temperatures.

of heating on binders containing natural rubber without carrying out a series of different tests. Figure 4 shows how the properties alter with heating at different temperatures.

The particular binder used consisted of $3\frac{3}{4}$ percent rubber (added as latex) in a bitumen/kerosene mixture. The presence of kerosene leads to more rapid breakdown of rubber than is found when either tar oil is used as a fluxing agent, or when no fluxing oil is used. The reduction in effectiveness, quantitatively, is therefore exaggerated. It can be seen that an increase in temperature above 160 C leads to a much more rapid rate of breakdown.

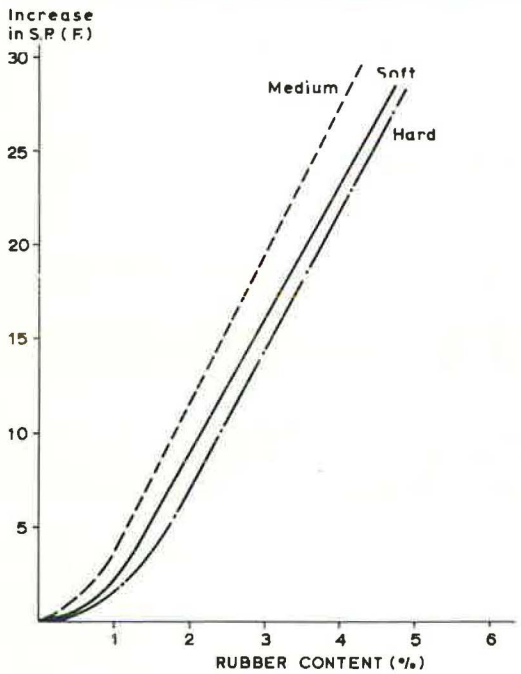


Figure 5. Effect of rubber concentration on softening point (ring and ball) in three Middle East asphalts using latex, unvulcanized rubber powder, and vulcanized rubber powder.

At the temperatures normally used for blending (around 160 C) the characteristics of rubber-asphalt vary rapidly during the first 2-3 hours. Initially, an increase in effectiveness (and, for example, elasticity) can be seen. In the first part of the heating process, which covers the normal blending time, the properties can change rapidly. This may explain some of the variability and inconsistent results that arise when different samples are compared in the laboratory. Until these changes have been explored for each type of rubber, comparisons between different rubbers may lead to incorrect conclusions. In practice, the amount of heating to which road binders are subjected before they reach the road is much greater than the period of "rapid change."

## Influence of Base Asphalt

The sources and types of asphalts used in the United Kingdom are less varied than in many other countries, in particular the United States. For the United Kingdom there is therefore less need to establish how rubber reacts in a variety of asphalts. Smith (1) investigated the effect of natural

rubber on three different asphalts from the same source (Middle East) but varying in hardness. The viscosities of the asphalts used were 170 pen (soft), 120 pen (medium), and 55 pen (hard). Some results of this investigation are reproduced in Figure 5. It is evident that, although there is some variation in the change in softening point between the three asphalts, it is marginal. Other researches have found much greater variation. It would seem necessary, therefore, for some simple tests to be carried out in the laboratory before asphalts not previously used with rubber are modified in this way.

## Results of Standard Tests on Rubber-Asphalt Blends

The properties of rubber-asphalts vary, depending on a number of factors, including the form of rubber and source of asphalt. Detailed results can therefore only apply to the particular circumstances of individual tests. Nevertheless it is possible to draw general conclusions from a wide range of test results, and the results quoted have been found to be typical for asphalts commonly used in the United Kingdom.

Softening Point—The softening point (ring and ball) invariably increased by the addition of natural rubber. Table 1 gives some typical results for various forms of rubber in a Middle East asphalt. On the average, the softening point of a rubber-asphalt blended with latex is raised by 3 C, with unvulcanized powder by $2\frac{1}{2}$ C, and with vulcanized rubber by $1\frac{1}{2}$ C for every 1 percent of natural rubber.

Individual tests on solid rubber (smoked sheet) dissolved in kerosene prior to preparing cut-back bitumen indicate that smoked-sheet-asphalt mixtures behave in a way similar to unvulcanized rubber powder.

Viscosity and Temperature Susceptibility—The addition of rubber has a marked effect on the viscosity of asphalt and, more importantly, on the rate of change of viscosity with temperature (i. e., the temperature susceptibility). At high road temperatures rubber increases the viscosity while at low temperatures the viscosity is reduced by rubber addition.

The method used for assessing the temperature susceptibility of rubber-asphalts (1) was that described by Lee et al (3), termed the logarithmic temperature coefficient. This is defined as

$$\log (\text{temp. coef.}) = \frac{\log \eta_1 - \log \eta_2}{\log T_2 - \log T_1}$$

where $\eta_1$ and $\eta_2$ are the viscosities measured at temperatures $T_2$ F and $T_1$ F.

As with the other rheological properties of rubber-asphalts, the temperature susceptibility varies with the amount of effective rubber in the binder. Figure 6 shows the progressive increase in susceptibility with reduction of effective rubber for latex and unvulcanized rubber blended into a 300-pen asphalt.

Elasticity—The most obvious change in asphalt when rubber is added is the marked elasticity that results. Asphalts (apart from blown asphalt) do not normally exhibit

TABLE 1

AVERAGE INCREASE IN SOFTENING POINT (RING AND BALL) IN °C WHEN 2 PERCENT AND 4 PERCENT OF DIFFERENT FORMS OF NATURAL RUBBER ARE ADDED TO A MIDDLE EAST ASPHALT

| Asphalt | Latex | | Unvulcanized Powder | | Vulcanized Powder | |
|---|---|---|---|---|---|---|
| | 2 Percent | 4 Percent | 2 Percent | 4 Percent | 2 Percent | 4 Percent |
| 300 pen. | $8\frac{1}{2}$ | — | 7 | — | 3 | — |
| 150 pen. | 5 | 13 | 4 | 9 | 2 | 4 |
| 100 pen. | 7 | 15 | 5 | 12 | 3 | 9 |
| 50 pen. | 4 | 12 | 5 | 12 | 4 | 8 |
| Average | 6 | 13 | 5 | 11 | 3 | 7 |

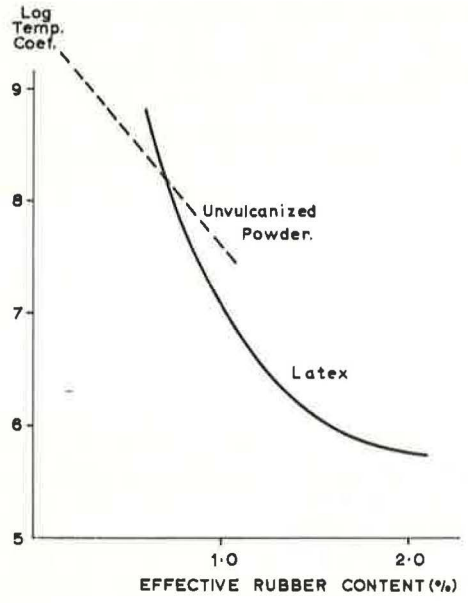


Figure 6. Variation of temperature susceptibility and effective rubber content of rubber-asphalt with latex and unvulcanized rubber powder.

noticeable elastic properties. With even small amounts of natural rubber (less than 1 percent), it often possible to see the elastic recovery when a sample is deformed at room temperature. Figure 3 shows how, even after two days heating at 140 C (284 F), the elastic properties are still appreciable. The very high values recorded when latex is used is accentuated by a gel structure that does not disappear until some time after initial dispersion of the rubber at normal blending temperatures.

Brittle Properties—Investigations into the brittle properties of rubber-asphalts carried out during the cooperative research program in the United Kingdom have been reported elsewhere (1). The properties were assessed by measurement of the extensibility of the binders at -10 C, and some results are reproduced in Figure 7. The extensibility measurements were carried out in a Hounsfield tensometer. The rate of extension was 0.1 in./sec. The load/extension data were obtained from photographic records of the tests at a film speed of 64 frames/sec. As with the other properties considered, the effects of unvulcanized rubber in latex and powder form are of similar magnitude. Vulcanized rubber is rather less effective. The broken lines in the diagram show linear relationships between increase in extensibility and in log (viscosity at 25 C), which are dependent on the base asphalt but independent of the form of rubber.

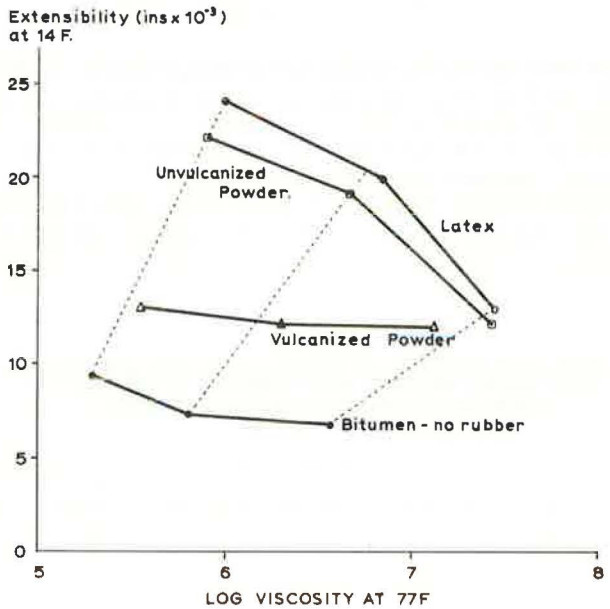


Figure 7. Relation between low-temperature (-10 C) extensibility and viscosity at 25 C after heating for 1 and 4 hours at 170 C.

Low-temperature measurements are very dependent on complete blending of the rubber. Undissolved rubber can form centers of discontinuity in the material at which stress concentrations occur, leading to premature failure.

### Properties of Rubber-Asphalts

The results of the standard tests referred to indicate how rubber addition alters the properties of asphalt. The important alterations can be summed up as follows:

1. Elastic properties are imparted to asphalts resulting in considerable departure from Newtonian flow;
2. The temperature susceptibility of viscosity is reduced; and
3. Brittleness is reduced.

When the modified binders are used in surfacing materials, advantages are gained that result from these changes in properties. They are improved resistance to cracking, resistance to fatting-up, resistance to flow or deformation (i. e., greater stability), and improved adhesion and resistance to stripping (arising from the improved "tack" of rubber). Although it is possible to carry out simulative tests to show these advantages, the most convincing demonstration is one actually carried out on the road.

## FULL-SCALE ROAD EXPERIMENTS

In the United Kingdom, the road experiments concerned four main types of surfacings: seal coats or surface-dressing, rolled asphalt, mastic asphalt, and open-textured carpets (bitumen macadam). Trials of other materials, such as fine cold asphalt, have been laid commercially and although successful have not been controlled systematically. The experiments carried out during the research have been reported up to 1963. They are summarized next, with additional data obtained subsequently.

### Seal Coats or Surface-Dressing

Experiments using asphalt cut-backs for surface-dressing have been carried out in various parts of England (4). This treatment, in which asphalt is sprayed onto the road and covered with single-sized chippings, is vulnerable to major temperature and weather changes. Although the viscosity of the sprayed binder is generally lower in the spring and fall than in the summer, it is not changed at short notice with the prevailing weather conditions. Sudden changes in the weather, therefore, may result in serious failures. The two main causes of trouble are the onset of very hot weather and heavy rain shortly after the work has been finished. Hot weather may cause failure in months, or even a year afterwards.

Natural rubber addition to asphalt has been shown to reduce considerably the susceptibility of the seal to both these types of failure. Eleven experiments were laid using rubber-asphalt as the binder; the rubber content varied between 1 and 2 percent by weight. Most of the binders were blended with vulcanized rubber, but in the last two latex was used. Full-scale spraying trials were also carried out to determine the maximum rubber content that could be used in practice.

From the series of experiments the following conclusions were drawn:

1. Rubberized binders resist fatting-up under heavy traffic (Fig. 8 shows a marked difference in the condition of a rubberized and a normal seal after hot weather).
2. The use of rubberized binders is of most benefit on heavily trafficked roads. On very lightly trafficked roads, rubber addition did not extend the life of the dressing. This result is to be expected because the most important change in properties obtained by the addition of rubber affecting this treatment is the reduction in temperature susceptibility. During warm weather and under traffic the binder rises up around the stone so that individual chippings are well-embedded. Rubber-asphalt resists this action, so that, whereas on heavily trafficked roads fatting-up is resisted, on lightly trafficked roads there is little tendency for fatting-up to occur.
3. There is a noticeably increased adhesion of the stone to the asphalt with rubberized binders, giving greater stone cover. In one particular experiment in

Figure 8. Fatting-up of normal asphalt seal after a spell of hot weather; the rubberized seal in the foreground has been little affected (rates of spread 4½ sq yd/gal).

Birmingham, the rubberized sections were immediately recognizable shortly after the work was completed by the lack of stone lying in the gutter—the gutters of the non-rubber sections were filled with chippings that had been whipped off. In another experiment in Cambridgeshire, following 24 hours of rain, the cover was very much greater on the rubberized sections.

The use of rubber for this type of treatment may be regarded as an insurance against failure. It is, of course, perfectly possible to obtain long lives for normal seals, but rubberized seals are more tolerant of adverse conditions.

Results from spraying trials show that the maximum "effective" rubber content of the binder that can be sprayed is ¾ percent when whirling-spray type jets are used. Since there will be some reduction in effectiveness during the blending process, this limits the amount of rubber that can be added to about 1 percent if latex is used and 2½ percent if vulcanized rubber is added. In most countries slotted jets are normally used, and with this type of jet the figures quoted may be doubled.

As was expected, there has been no observable difference between the behavior of rubberized binders blended with natural latex and natural rubber powders, provided the binders had approximately the same effective rubber content.

## Dense Mixed Materials

The two types of dense materials mainly used in the United Kingdom are rolled asphalt and mastic asphalt. Six full-scale experiments have been laid to assess the properties of rubberized rolled asphalt, and four to assess the properties of rubberized mastic asphalt.

Not all of these experiments have reached the end of their useful life, since the materials would normally be expected to last 15 years or more. It has, however, been possible to draw firm conclusions from the behavior of the materials so far.

TABLE 2

RESULTS AFTER 4½ YEARS OF A FULL-SCALE ROAD EXPERIMENT IN LEICESTERSHIRE, TO ASSESS THE CRACK RESISTANCE OF RUBBERIZED ROLLED ASPHALT OVER CONCRETE

| Material | Thickness (in.) | No. of Cracks in Surfacing | Total Joints or Cracks in Concrete | Percent Cracking |
|---|---|---|---|---|
| Normal rolled asphalt | 5 | 14[a] | 32 | 44 |
| Rubberized rolled asphalt | 5 | 0[a] | 31 | 0 |
| Rubberized rolled asphalt | 4 | 0 | 23 | 0 |
| Rubberized rolled asphalt | 3 | 2[b] | 22 | 9 |

[a] Plus one very slight crack.

[b] Plus two very slight cracks.

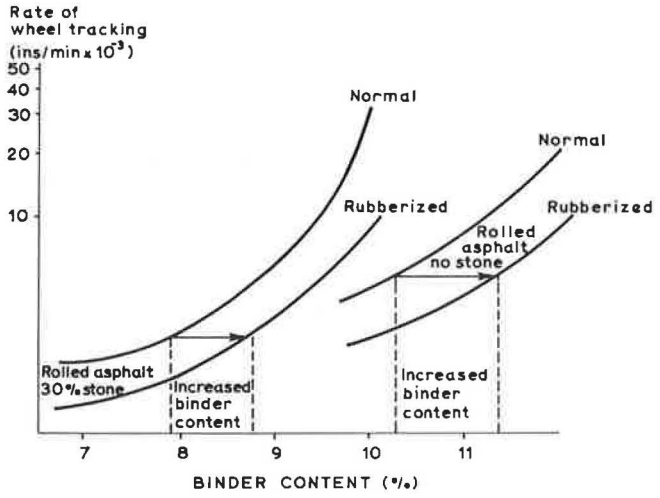


Figure 9. Rate of wheel-tracking of rolled asphalt with and without rubber.

Natural rubber added to rolled asphalt considerably improves the crack resistance of the material. As an example, the results to data of an experiment designed to assess this property are given in Table 2. In this experiment, the total thickness of the surfacing courses was varied to establish whether a lesser thickness would be successful in reducing cracking while at the same time saving costs. It is evident that not only is cracking reduced but the thickness can be reduced by 20 to 30 percent. Other experiments described previously (4) have confirmed the improved crack resistance of rubberized materials of this type.

It has also been evident that deformation is reduced by the addition of rubber. To assess the extent to which the stability of rolled asphalt is increased by rubber addition, mixtures were investigated using the wheel-tracking test (5). Test results are shown in Figure 9. Two mixtures were tested, a rolled asphalt (to British Standard BS 594) with 30 percent stone and a rolled asphalt with no stone. With both mixtures, stability was significantly increased by 3 percent natural rubber, and it was found that an extra 1 percent of binder could be used without loss of stabliity. This additional binder itself increases the crack resistance.

The improvements in properties of mastic asphalt by rubber addition are similar to those found for rolled asphalt. The full-scale experiments have again been confirmed in situations where cracking is a problem—e.g., over joints between concrete slabs.

In mastic asphalt laid at about 200 C (392 F) or more, greater percentages of rubber are used. It is normal to add 15 percent vulcanized rubber or 7½ percent unvulcanized rubber by weight of soluble binder. Table 3 gives the results of one experiment showing how resistance to cracking is increased by the rubber.

TABLE 3

RESULTS AFTER 7 YEARS OF A FULL-SCALE ROAD EXPERIMENT TO ASSESS THE CRACK-RESISTANT PROPERTIES OF RUBBERIZED MASTIC ASPHALT OVER CONCRETE

| Type | Binder Content (percent of mastic mortar) | Rubber Content[a] (percent of soluble asphalt) | No. of Joints | No. of Cracks |
|---|---|---|---|---|
| Plain mastic to BS 1447 | 13½ of 65 pen. | 0 | 9 | 7 |
| | 15 of 40 pen. | 0 | 9 | 8 |
| | 13½ of 40 pen. | 0 | 9 | 7 |
| Plain mastic with expanded metal | 13½ of 40 pen. | 0 | 8 | 7 |
| Rubberized mastic asphalt | 12 of 65 pen. | 16½ | 8 | 3 |
| | 12¾ of 40 pen. | 17½ | 8 | 4 |
| | 15½ of 25 pen. | 18 | 8 | 0 |
| Stone-filled mastic[b] | 15 of 40 pen. | 0 | 8 | 3 |

[a] Vulcanized rubber powder.
[b] 40 percent of ⅜-in. granite chippings in the mastic mortar.

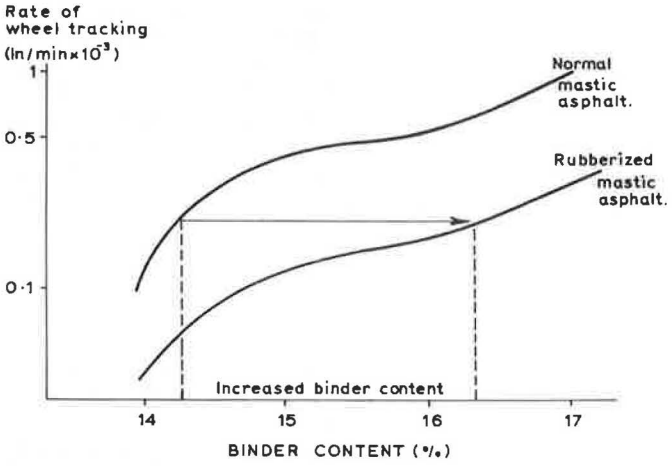


Figure 10. Rate of wheel-tracking with and without rubber.

The stability of mastic asphalt is also increased by rubber addition and an investigation using the wheel-tracking test, similar to that carried out on rolled asphalt, was made with mastic asphalt. As can be seen in Figure 10, the increase in stability of mastic asphalt is even more marked than that of rolled asphalt, and with 4½ percent unvulcanized rubber added to the binder, an increase of 2 percent in the binder content does not impair the stability of the mixtures.

## Open-Textured Bitumen Macadam

The normal life of open-textured carpets of bitumen macadam is only 5 to 6 years. An increased life of only 1 or 2 years will more than justify the extra cost of adding rubber to the material. Two experiments have been started using open-textured materials. The first was completed a few years ago; the second has yet to be completed.

In the first the asphalt content of the mixed material was varied beyond the limits allowed in the British Standard (BS 1521) to see if, by rubber addition, leaner mixtures could be used. The experiment showed that, although there was no advantage with binder contents much below those allowed in the Standard, up to 50 percent increased life was obtained when 4 percent of rubber was added to the normal materials. The results of the experiment are given in Table 4. The second experiment still has a few years to run, but appears so far to confirm those results.

TABLE 4

RESULTS AFTER 8 YEARS OF A FULL-SCALE ROAD EXPERIMENT TO ASSESS THE INCREASE IN LIFE OF BITUMEN MACADAM BY THE ADDITION OF UNVULCANIZED RUBBER POWDER

| Binder Viscosity | Binder Content (percent) | Useful Life (years) | | Increase in Life With Rubber (percent) |
|---|---|---|---|---|
| | | Without Rubber | With Rubber | |
| Asphalt cutback 100 sec. at 40 C (S. T. V.) | 2.75 | 4 | 4 | 0 |
| | 3.0 | 5 | 5 | 0 |
| | 3.5 | 5 | 7 | 40 |
| | 3.75[a] | 6 | 8 | 33 |
| | 4.25[a] | 5 | 7 | 40 |
| 300 pen. | 2.75 | 6 | 6 | 0 |
| | 3.0 | 6 | 8 | 33 |
| | 3.5 | 6 | 8 | 33 |
| | 3.75[a] | 6 | FG[b] | 50[c] |
| | 4.25 | 6 | FG[b] | 50[c] |

[a]Only these binder contents within British Standard BS 1621.
[b]Sections still fairly good after 8 years.
[c]Life taken as 50 percent increase assuming the surfacings would be replaced the following year.

TABLE 5

BLENDING TIMES FOR UNVULCANIZED AND VULCANIZED NATURAL RUBBER IN ASPHALT

| Temperature | | Blending Time | |
|---|---|---|---|
| C | F | Unvulcanized Rubber | Vulcanized Rubber |
| 180 | 356 | * | ½ hr |
| 170 | 338 | * | 1 hr |
| 160 | 320 | 10 min | 2 hr |
| 150 | 302 | 15 min | 3-4 hr |
| 140 | 284 | 25 min | 6-8 hr |
| 130 | 266 | 35 min | * |
| 120 | 248 | 45 min | * |

*Not recommended.

## BLENDING OF RUBBERIZED BINDERS

Although the establishment of the properties of rubberized asphalt and the advantages to be gained from their use has formed a major part of the research, devising the best methods of using rubber has also been an important part of the work. This investigation has fallen into two parts: the formulation of the best blending techniques, and the recommendation of any alterations in practice necessary for the successful laying of rubberized materials.

Rubberized asphaltic binders may either be blended prior to their use in mixed materials or, in most cases, the rubber may be added during the manufacture of the mixtures (2). For seal-coats the binder must be pre-blended. The rules for pre-blending the binders vary with the form of rubber. Rubber powders are mechanically dispersed throughout the volume of asphalt and heating is continued for sufficient time at the blending temperature for the rubber to be completely dissolved in the binder (Table 5). The time differs for vulcanized and unvulcanized rubber. It is important that all the rubber be dissolved. Undissolved rubber will not contribute to the improved properties of the asphaltic materials.

In latex form, rubber requires a slightly different technique because the relatively large quantities of water stirred into hot asphalt can be dangerous if the correct procedure is not followed. A simple method is to spray the latex onto the surface of the hot asphalt, allowing about 20 sec for most of the water to be vaporized before the rubber is drawn into the bulk of the asphalt. An antifoaming agent may be used or a tank of sufficient capacity used to allow for the increase in volume occurring when the small residual quantity of water expands as steam in the asphalt. Blending times and temperatures are the same as those given in Table 5 for unvulcanized rubber.

Both latex and unvulcanized rubber powder may be added to the mixed asphaltic materials during manufacture. Vulcanized rubber should not be added at this stage, but must be pre-blended into the asphalt. Rubber powder is poured into the mixing chamber during the injection of the liquid asphalt; latex is sprayed in as soon as the asphalt has wetted the stone. An additional mixing time of 5 to 10 sec is generally allowed for mechanical dispersion of the rubber throughout the mix, although this period will depend on the mixer. Completion of the blending process occurs during transport of the material to the site of laying. A minimum of ½ hour before laying is generally considered advisable.

Direct addition of rubber to the mixer may be used for materials mixed above about 135 C (275 F). Below this temperature, however, the minimum blending time cannot generally be guaranteed and pre-blending is desirable. At mixing temperatures below about 120 C (250 F) pre-blending is essential.

## LAYING RUBBERIZED MATERIALS

Rubberized asphaltic materials are laid by the same techniques as the normal equivalent materials. The only practical difficulties that may be met arise from the

increased viscosity of the asphalt and the increased tack. Rubberized materials are "tougher" to handle. To overcome these difficulties the mixing and laying temperatures recommended are 10 to 15 C (18 to 27 F) higher than would normally be used. Rolling temperatures should also be about 10 C (18 F) higher than normal. If these rules are followed, no difficulties should be encountered.

## CONCLUSIONS

The properties of asphalt are modified by the addition of natural rubber in the following ways:

1. A marked elasticity is imparted to the asphalt and there is considerable departure from Newtonian flow;
2. The temperature susceptibility of viscosity is reduced; and
3. Brittleness is reduced.

These alterations occur simultaneously.

As a result of these changes rubberized materials have shown advantages in full-scale road experiments. The advantages vary depending on the type of road surfacing. In surface-dressing or seal coats rubber prevents or markedly reduces fatting-up in hot weather and a greatly increased initial chip retention has been shown, particularly when wet weather follows shortly after treatment. In both types of dense materials—rolled asphalt and mastic asphalt—the resistance to cracking is much increased as is the stability or resistance to deformation. In open-textured bitumen macadam carpets an increased life of up to 50 percent has been found.

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# Effect of Water on Behavior of Sand-Asphalt Mixtures Under Repeated Loading

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This paper reviews the fundamental concepts related to adhesion and the stripping phenomenon in bituminous mixtures. Stripping, which may occur in only a small portion of the pavement, is considered a great economic loss and an engineering failure in terms of proper mixture design.

The factors affecting adhesion and adhesion failure are innumerable. They include the material characteristics, construction techniques, and diversified environmental conditions. Due to the extreme complexities of adhesion and adhesion failure phenomena in bituminous mixtures, a specific engineering problem is analyzed in this report and the validity of the adhesion failure concepts is discussed.

In the study, a hot-mix sand-asphalt mixture that had shown severe distress and disintegration in the field is subjected to numerous laboratory tests to evaluate the causes of stripping. The results indicated that standard test methods for effect of water on cohesion of compacted mixtures did not reveal any stripping tendencies. However, the application of dynamic loading on saturated mixtures resulted in drastic reduction in the compressive strength. Similarly, the creep response of mixtures was affected by the number of load repetitions. It is also shown that the soaked specimens under dynamic loading exhibit permanent deformation that may lead to failure in the sand-asphalt mixture.

•ONE OF THE complex problems in the field of highway engineering, existing since asphalt paving technology came into existence, is stripping. The term stripping, as employed by highway engineers, denotes the occurrence of adhesion failure, or a weakening of the cohesive bonds within the asphalt-aggregate system. Although this phenomenon may occur in a small portion of the pavement structure, it is considered a great economic loss and an engineering failure in terms of proper mixture design.

The factors affecting the adhesion failure phenomenon are innumerable. They include the material characteristics, construction techniques, and diversified environmental conditions. Realizing that theories related to adhesion and adhesion failure are still debatable for even the most simplified cases, it becomes evident that no single theory can be postulated at this time to predict the stripping phenomenon in bituminous mixtures.

In this report, an attempt is made to find the causes of stripping observed in hot-mix sand-asphalt mixtures and to develop a realistic hypothesis and testing procedure for evaluation of these stripping characteristics.

## LITERATURE REVIEW

In an earlier report (1) of this investigation, the literature pertaining to the effect of water on bitumen-aggregate mixtures was reviewed and many theoretical and practical

Paper sponsored by Committee on Effects of Natural Elements and Chemicals on Bitumen-Aggregate Combinations and presented at the 48th Annual Meeting.

aspects of the subject were discussed. The result of this review can be summarized in three areas of scientific challenge, each with extreme complexity. They are understanding adhesion mechanism, revealing the modes of adhesion failure in adhesive-adherent systems, and developing evaluation techniques for material characterization. The study of the mechanism of adhesion, which encompasses mechanical interlock, molecular orientation, chemical reaction, and interfacial energy, is beyond the scope and capabilities of this research project. However, it must be pointed out that each of these theories have their merits and limitations and are strongly supported by theoretical concepts.

In the second area of scientific challenge, mechanics of adhesion failure, four basic hypotheses have been presented. They are displacement, film rupture, detachment theory, and pore pressure. Of course, it is obvious that owing to the complexities of material composition and the diversity of environmental conditions, no single mechanism may be adequate to explain the stripping phenomenon in bituminous mixtures. These concepts are briefly summarized as follows:

1. Displacement Concept—According to this theory, the binder-aggregate junction in the presence of water becomes thermodynamically unstable and retracts to a more stable position (2, 3, 4). It is generally believed that this displacement is a function of viscosity; i.e., high viscosity binders show higher resistance to displacement. However, it has been stated that, in order for displacement phenomena to be initiated in a mixture, the binder-aggregate interface should become exposed to the water phase. That is, well-coated aggregate particles may not exhibit any binder displacement unless the continuity of the aggregate coating is destroyed.
2. Detachment Concept—This theory attributes the adhesion to a thermodynamic replacement of the bitumen by a thin film of water that may come from either outside or from within the aggregate while the bitumen coating remains intact (5, 6, 7). The characteristics of the interface are believed to be very important in the detachment process. The water reaching the interface may become intimately associated with the lattice of the mineral surface.
3. Film Rupture Concept—This theory explains the initiation of displacement in a well-coated aggregate. It is believed that dynamic loading and the kneading action of traffic induce rupture at the sharp corners of aggregate where the binder film is thinnest (8, 9, 10). Once the binder is ruptured, water may easily displace bitumen on the aggregate surface, as previously discussed.
4. Pore Pressure Concept—It has been postulated that the buildup of pore pressure in mixtures of high void content may result in stripping phenomena (3, 11). That is, on a wet surface of bituminous pavement, additional forces due to traffic also act and these greatly exceed the thermodynamic forces. In a saturated pavement under dynamic load, water is pressed into the pavement in front of the moving load and sucked out behind the wheel contributing to the stripping phenomena.

Among these four concepts, the displacement and detachment theories can be classified as the primary causes of stripping and pore pressure; film rupture concepts are in fact only contributing to the phenomena.

The third step in the analysis of adhesion is the development of realistic evaluation methods. In the last 40 years a great number of laboratory techniques have been proposed for the evaluation of adhesion failure in bituminous mixtures (1).

Tests falling under the category of dynamic immersion tests, such as the Nicholson test (12) and the wash test (13), bear the basic criticism that the simulation effect of traffic loading is unrealistic. The agitation of loose mixtures in water does not resemble the behavior of compacted mixtures in the road.

The chemical immersion tests, such as the Riedel and Weber test (14), are also criticized due to the use of chemicals such as sodium carbonate in the evaluation of adhesion.

A great number of methods fall under the static immersion test categories where resistance of binder-aggregate systems to stripping is measured without inducing the dynamic effect of traffic. These tests, such as the Lee (4), Oberbach (15), and ASTM

(D 1664-64T) tests, employ a visual examination as a measure of stripping, and the results of evaluation have little relation to field performance.

Many tests, such as the cold water abrasion test (16), English trafficking test (3, 17), and test tracks (18), have been proposed to simulate traffic conditions. These methods, although they use a more realistic approach such as using actual paving mixtures instead of artificial material, are rather costly and beyond the capabilities of many organizations.

One type of widely used method for the evaluation of stripping characteristics is known as the immersion-mechanical test. Included in this type is the immersion-compression test, which will be discussed later. It has failed to duplicate road failures attributed to stripping. The basic criticism of these tests is the lack of traffic simulation.

Recently, a number of tests utilizing quantitative coating measurements have been greatly advocated. In these tests the percent of area stripped is measured through the use of adsorptive principles. The dye adsorption (19, 20) and the radioactive isotope tracer (21, 22) and the tracer salt are in this category. Althougth the technique of measuring the percent of stripping is a sound method, there is no indication that the stripping generated in these laboratory mixtures simulates the field condition. Furthermore, the knowledge of percent of stripping alone is of no practical value unless it is related to the mechanical properties used in the design of bitumen mixtures.

## ANALYSIS OF A PROBLEM

In this chapter, a specific engineering problem related to stripping phenomena is analyzed and the validity of the concepts previously postulated are discussed. The problem arises from the occurrence of severe distress and disintegration in two sections of newly constructed state highways in Florida. The detailed and careful examination of the failed pavement revealed beyond any doubt that distress was due to the loss of cohesive bond in the hot-mix sand-asphalt (SAHM). The study of the first pavement failure was on US 231 (Florida State Road 20) in Bay County, and it indicated that stripping resulted in complete loss of cohesive bond within the ½-in. layer of leveling course (Fig. 1). The construction reports indicate that this layer had been placed prior to undesirable climatic conditions, including a severe freeze. Due to the cold weather, the work had been discontinued for a considerable length of time. Later, as soon as the weather permitted, the uppermost leveling course and the surface were completed. Soon after construction, rutting and severe distress were reported in the paved sections.

The second road failure occurred on State Route 30 west of Fort Walton. It was also attributed to stripping in the SAHM. According to State Road Department reports, the

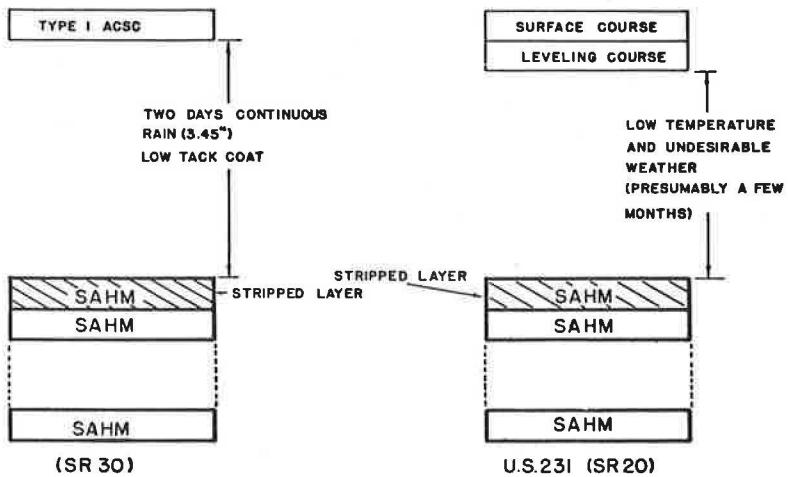


Figure 1. Stripping of SAHM mixture in relation to composition and weather.

construction of SAHM courses was completed by March 31, 1964. The construction of asphaltic concrete surface course type 1 (ACSC) was started on April 13, 1964; however, owing to severe rain (3.45 in. from April 13 to April 15), the paving operation was stopped for 2 days and resumed on April 15. The occurrence of stripping in certain sections of this newly constructed highway was extensively investigated by the Division of Materials, Research and Training, Florida State Road Department, and the results of their study are summarized as follows:

1. Materials used in the paving operation, such as sand and asphalt-cement, and the mix design all conformed with the SRD specifications.
2. The mixing temperature was within the specification limits.
3. Rainfall during the construction of SAHM was not significant.
4. The air temperature was apparently below 40 F during part of the time SAHM was being placed.
5. Tackcoats placed on old roadway between SAHM courses and between SAHM and type 1 ACSC did not meet the SRD specification.
6. Heavy rainfall was reported from April 13 to 15, prior to placing of type 1 ACSC.
7. Stripping tests conducted on the remixed stripped material from the roadway, the remixed samples from satisfactory areas, and the laboratory mixtures have not indicated any stripping tendency in the temperature range of 77 to 140 F.

Figure 1 shows the distressed layers of SAHM in relation to the climate and the construction. As is indicated, both layers were exposed to severe weather prior to being overlaid by another hot-mixed bituminous layer. Therefore, during the period of rain and undesirable weather, the exposed SAHM course became partially or fully saturated with water and the binder-aggregate bond became thermodynamically unstable. This is a logical deduction from the classical displacement concept of adhesion failure.

To test the validity of this concept as applied to bituminous mixtures, one expects that the use of the immersion-compression test, ASTM Method D 1075-54, should reveal the stripping tendencies of the sand-asphalt mixture. On the contrary, however, the results of this test on SAHM mixtures (23) indicate that this mixture does not exhibit any significant tendency toward stripping and in no case did the loss of compressive strength exceed 15 percent.

To search for the causes of stripping in sand-asphalt mixtures, a test method has been developed that includes the effects of dynamic loading. In the following sections this method of analysis is presented.

## MATERIALS AND TESTING PROCEDURE

### Materials

The materials used in this study were chosen to conform as closely as possible with those used in the two cases of field stripping observed. The aggregate was a fine sand obtained from the Okaloosa Company. Its gradation curve is shown in Figure 2. The bituminous binder selected for this investigation was a 60/70-penetration asphalt cement with a laboratory designation AS 65-1, and is similar to the material used in the construction of the SAHM courses. The test characteristics of the asphalt cement and the sieve analysis on the aggregate indicate that they meet Florida State Road Department specifications. The results of tests on the asphalt cement are given in Table 1.

TABLE 1

PROPERTIES OF ASPHALT CEMENT AS 65-1

| Property | Value | Property | Value |
|---|---|---|---|
| Penetration | 60 | Viscosity at 275 F, poises | 440.51 |
| Softening point | 129 F | Viscosity at 210 F, poises | 5908.5 |
| Ductility | 150+ | Viscosity at 140 F, poises | 3094.55 |
| Specific gravity | 1.008 | TFOT viscosity at 140 F, poises | 7031.91 |
| Viscosity at 77 F, poises | $3.16 \times 10^6$ | Viscosity ratio | 2.28 |

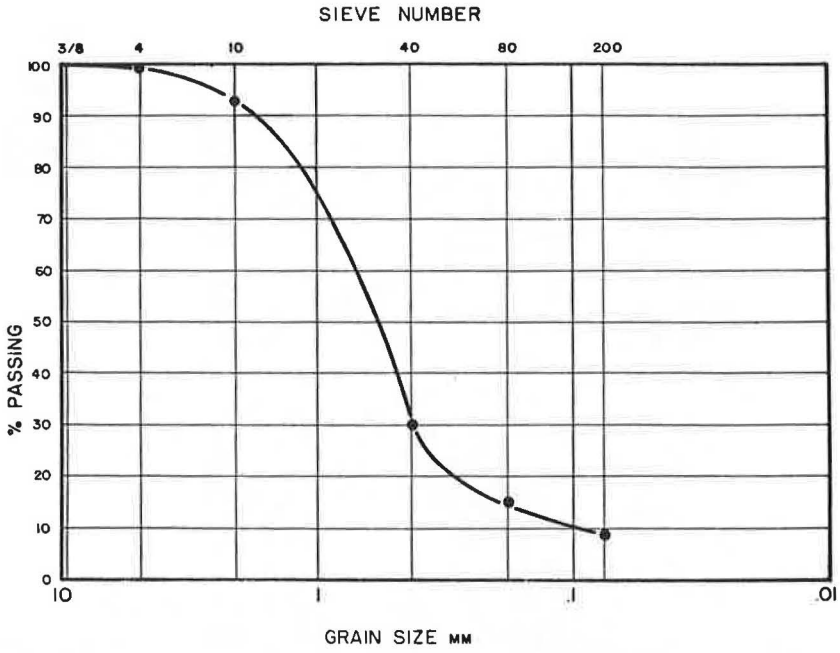


Figure 2. Gradation curve of sand used in hot-mix sand-asphalt.

## Sample Preparation

Mixing of the sand-asphalt was done according to ASTM D 1074-60, except that a batch consisted of enough material to compact 6 specimens. It was felt that a 6-sample batch would insure better uniformity of the mix for such small samples. Mixing temperature was approximately 325 F, and was maintained above a minimum of 270 F by using infrared lamps. The batch was then divided into 6 parts and oven-stored at 270 F for compaction. All samples were compacted within 2 hours after mixing.

To compact the samples, Harvard miniature molds, 1.312 by 2.624 in., were heated in a water bath, removed, dried, and oiled. The mold was then placed over a bottom plunger and approximately 55.2 grams of the sand-asphalt mixture, which was found to produce a sample 1.312 in. high, was introduced in 2 layers. Each layer was spaded 25 times, and the top plunger was placed on the mix. Compaction was done using the double plunger arrangement and a static load of 3000 psi maintained for 2 minutes. The specimen was then ejected, marked for identification, and oven-cured at 140 F for 24 hours. After removal from the oven, the samples were allowed to air cool for 1 hour. The average height and bulk specific gravity of each sample was then determined and they were stored in dark, carbon-dioxide filled cans at room temperature until needed for testing.

Sample Saturation—An attempt was made to saturate the samples according to ASTM D 1075-54; however, it was found that after 24 hours in the 140 F water bath, the samples had only 50 percent of their air voids filled. A vacuum process was therefore developed that used a small plastic cap on one end of the sample and a rubber membrane surrounding the sample. It was found that approximately 18 percent of the original 20 percent air voids were filled with water after only 15 seconds. All samples were therefore saturated by applying a 30-mm Hg vacuum pressure for 30 seconds. This was sufficient to draw double-distilled water entirely through the sample. After saturation, the membrane was sealed to prevent loss of moisture, and was tested within 22 ±2 hours from the time of saturation. This was necessary to insure that the stripping action of the water on the sand-asphalt mixture was uniform for both conditioned and unconditioned samples.

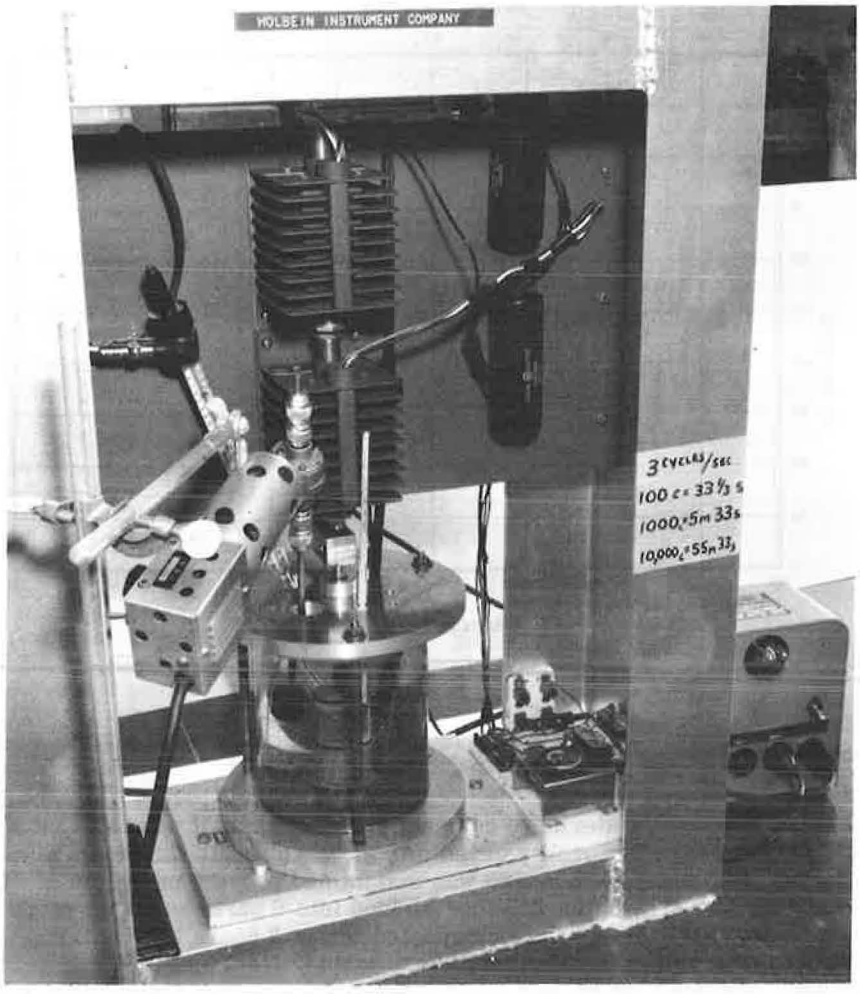


Figure 3. Dynamic preconditioning apparatus.

Preconditioning—Preconditioning of the samples consisted of applying a variable number of loads to the sample at a controlled temperature of 140 F. This was done to simulate the action of traffic on the hot, saturated sand-asphalt layer. It was accomplished using a Skinner Polynoid Linear Actuator attached to a frame and controlled by a timer giving a 3-cps frequency of load application. The load was 7.7 lb (5.695 psi) and lasted approximately 0.04 second.

To precondition a specimen, it was placed in a water bath at 140 ± 5 F for 2 hours to reach the test temperature. It was then transferred to an environmental chamber maintained at 140 ± 1 F. The chamber was adjusted, and the proper number of loads applied. Figure 3 shows the preconditioning apparatus. After preconditioning, the sample was again sealed and placed in a 77 F water bath for at least 1 hour before further testing was carried out.

## Unconfined Compressive Strength Test

Unconfined compression tests were run on a number of specimens. At least 3 samples were tested at each condition of preconditioning and saturation, and the compressive strength was reported as the average value. The test was conducted at 77 F, and the compressive strength machine operated at a rate of 0.08 in. per minute.

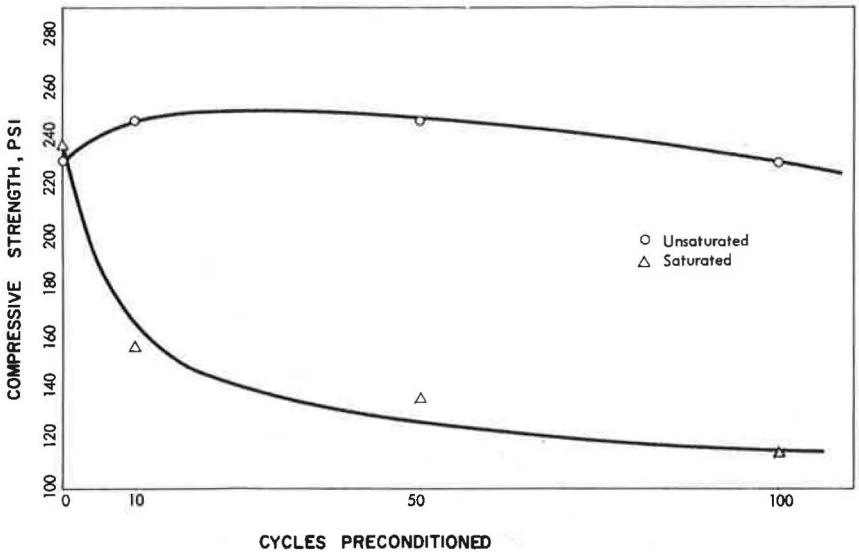


Figure 4. Effect of preconditioning on compressive strength of sand-asphalt.

## ANALYSIS AND DISCUSSION OF RESULTS

### Effect of Soaking

As indicated, unconfined compression tests were conducted on saturated and unsaturated specimens. The results of compressive strength tests on samples prepared without any dynamic preconditioning indicate that no significant differences occur among saturated and unsaturated samples (Table 2). The same conclusions have been reached from previous studies (23) in which the ASTM standard method of soaking was followed. Therefore, based on previous and present data, it can be concluded that, for the materials studied, the soaking alone will not result in any significant reduction in the compressive strength of the mixtures.

### Effect of Repeated Loading

The saturated and unsaturated specimens were subjected to dynamic preconditioning. The data in Table 2 and Figure 4 indicate that the compressive strength of unsaturated preconditioned samples is not significantly affected by the load repetition. In fact, the compressive strength is slightly increased. This is believed to be due to some densification in the mixture. Also in Figure 4, the compressive strength of saturated samples is plotted for various numbers of load applications. As indicated, the dynamic preconditioning drastically reduces the cohesive strength of the mixtures.

The induced strain in the specimens during preconditioning is also an important parameter that should be considered in the analysis of data. To measure these induced strains, an LVDT was mounted on the dynamic preconditioning apparatus (Fig. 3) and the strains were recorded. Figure 5 shows the relative deformation of soaked and unsoaked specimens under repeated loading. Soaked specimens have considerably higher strain than unsoaked specimens. In fact, the soaked specimens failed after 3000 cycles, whereas the unsoaked samples showed only a slight bulging at this loading condition. Figure 6 shows

TABLE 2
EFFECT OF DYNAMIC PRECONDITIONING ON COMPRESSIVE STRENGTH

| Cycles of Dynamic Loading | Compressive Strength (psi) | |
|---|---|---|
| | Unsaturated Specimen | Saturated Specimen |
| 0 | 229.6 | 235.6 |
| 10 | 244.5 | 151.8 |
| 50 | 245.6 | 136.0 |
| 100 | 229.3 | 114.2 |

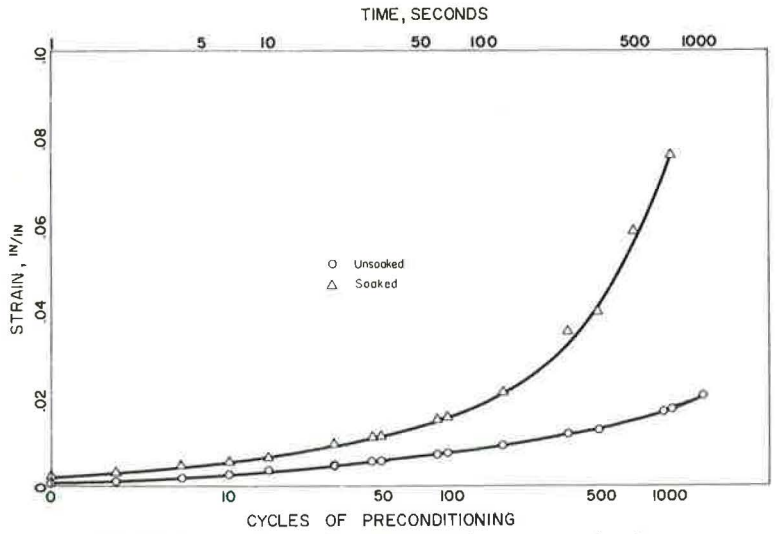


Figure 5. Induced deformations under dynamic loading.

soaked and unsoaked samples after 500 and 3000 cycles. Samples 1 and 2 are saturated specimens corresponding to 3000 and 500 cycles respectively; samples 3 and 4 are unsaturated samples tested under 500 and 3000 cycles respectively. Comparison of specimens 1 and 4 as well as 2 and 3 shows the differences in the deformation response of saturated and unsaturated samples.

## Mechanism of Stripping

In the previous section it was shown that the repeated application of axial load results in the reduction of strength of saturated sand-asphalt mixtures. This strength reduction can be attributed to two separate mechanisms, namely, mechanical and physicochemical degradation. In mechanical degradation phenomena, it can be postulated that the repeated load application may result in formation of micro-rupture planes in the bituminous binder, thus reducing the strength of the specimen. On the other hand, the physicochemical degradation is based on the thermodynamic instability in the binder-aggregate interface. In order to investigate this phenomenon in the laboratory, two series of identical specimens, one containing antistripping agents, were prepared and subjected to dynamic preconditioning. The antistripping agent X was added in the amount of 2 percent of the weight of bituminous binder. These specimens, saturated and unsaturated, were then subjected to 0, 10, 50, 100, and 300 cycles of repeated loading as described previously. After preconditioning, the compressive strengths of both saturated and unsatureated, specimens were determined and the corresponding compressive strength ratio was

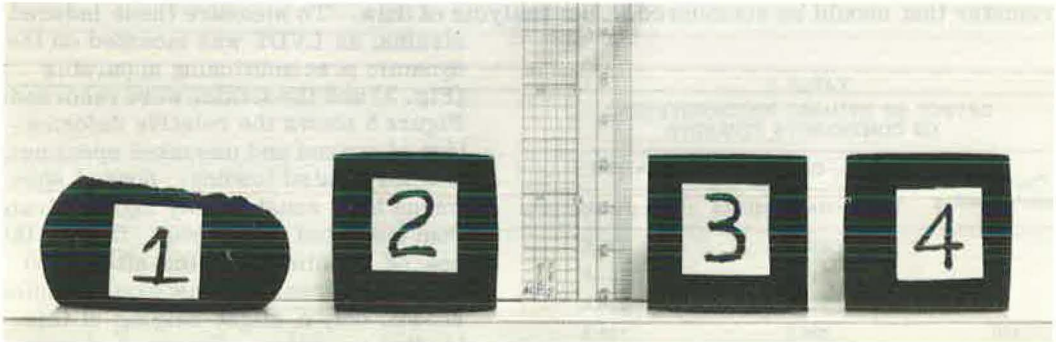


Figure 6. Physical condition of soaked and unsoaked samples after dynamic preconditioning.

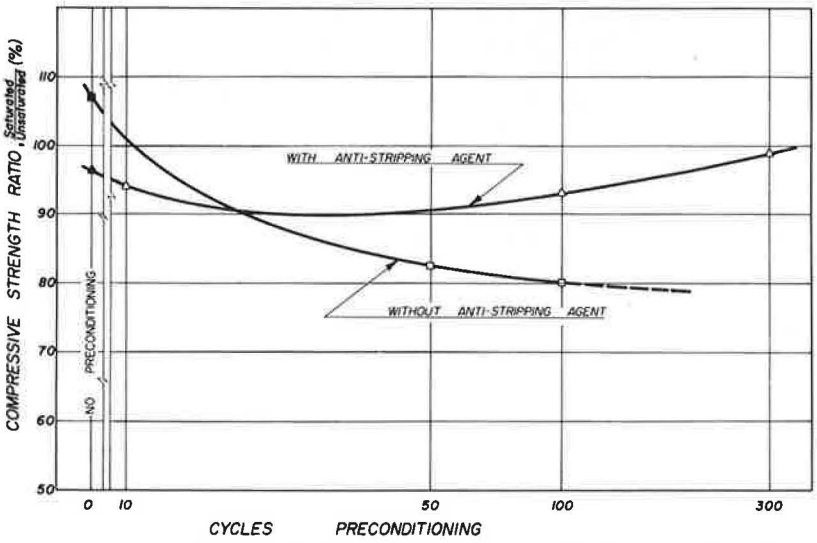


Figure 7. Effect of anti-stripping agent on the strength of saturated samples.

calculated. Figure 7 shows the results of these tests. As indicated, the samples without antistripping agents exhibit strength reduction, whereas the corresponding specimens with antistripping agents are not affected by the repeated load application. This observation can be considered as the supporting evidence for the presence of stripping phenomena in the bituminous mixture studied. It could also be utilized as a protective measure to prevent the recurrence of stripping failure in sand-asphalt mixtures.

## Effects of Mixture Density

There are primarily three prerequisites for the occurrence of stripping phenomena in bituminous mixtures: presence of water in pavement, repeated load application, and the physicochemical nature of the bituminous-aggregate system. In order to eliminate or reduce the chances of stripping, one should attempt to eliminate at least one of these prerequisites. From the pavement design point of view, the water present in the pavement can be reduced by decreasing the void content of the bituminous mixture. In fact,

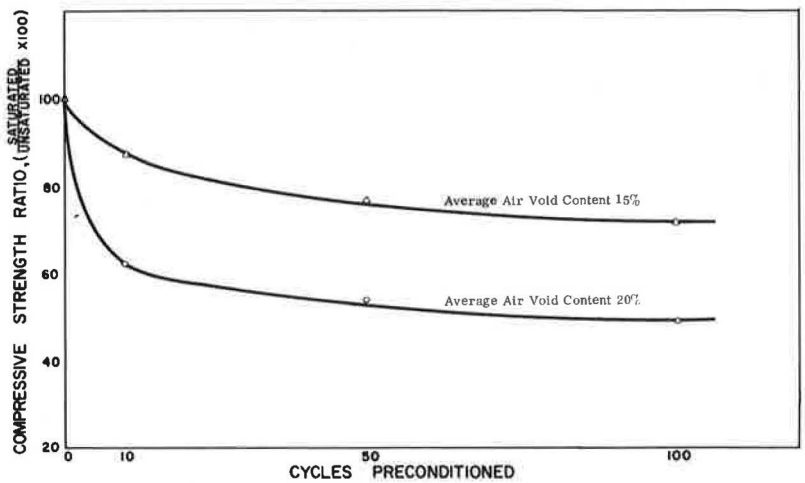


Figure 8. Effect of preconditioning on compressive strength of sand-asphalt.

this approach could be considered as one of the preventive measures in the construction of bituminous paving using the physicochemically unstable bituminous-aggregate system. To investigate the validity of this argument, the strength characteristics of sand-asphalt mixtures with two different void contents were compared. These specimens were subjected to identical saturation, dynamic preconditioning, and unconfined compression tests. The compressive strength ratios of these two series of specimens are shown in Figure 8. It is quite evident that when the air void content was reduced from 20 to 15 percent, the strength of the saturated mixture improved considerably. In this study the void content was reduced by increasing the compactive effort. However, it is believed that similar results could be achieved by increasing the percentage of bituminous binder in the mixture, i.e., by increasing the thickness of the binder film in the sand-asphalt mixture.

## SUMMARY AND CONCLUSIONS

In this investigation the compressive strength and creep response of saturated and unsaturated sand-asphalt specimens were investigated. The previous laboratory analysis had indicated that present test methods cannot duplicate the excessive loss of cohesion of these mixtures as observed in the field. To duplicate the field conditions, a dynamic preconditioning method was devised in which the samples were subjected to a specified number of load repetitions prior to testing in unconfined compression. The following are the conclusions of this research:

1. The compressive strength of saturated and unsaturated samples without dynamic preconditioning does not differ significantly.
2. The compressive strength of unsoaked samples is not significantly affected by the number of cycles of dynamic preconditioning. Owing to possible densification, a slight increase in the strength is observed at intermediate cycles of preconditioning.
3. The compressive strength of soaked samples is drastically reduced by the dynamic preconditioning.
4. The unsoaked samples under dynamic loading exhibit some permanent deformation. The soaked samples, however, show a much greater permanent strain and approach to the state of failure at a lower number of load repetitions.
5. Strength reduction due to dynamic preconditioning under axial loading can be improved by using an antistripping agent. It is postulated that physicochemical efforts (stripping) are responsible for the reduction of strength of saturated sand-asphalt mixtures.
6. Reduction in the percent air voids in the samples improves the resistance of the bituminous mixture to water. Reduction in void content either by compaction or by change in mixture composition can be used as a preventive measure in the field.

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