This paper describes a research study of the flexural and fatigue properties of both gap-graded and continuously graded bituminous mixtures to establish the factors contributing to fatigue and the conditions under which gap-graded mixtures can be used to maximum advantage. Laboratory-prepared specimens and in situ measurements on a full-scale experimental road pavement were used to study the factors involved in the fatigue subsystem. In the laboratory tests an electrodynamic system of impulse loading was used on trapezoidal-shaped specimens. High-frequency seismic and ultrasonic methods emerged as valuable nondestructive techniques for the measurement of elastic moduli both in the field and in the laboratory. Correlations between measured strains and deflections within the experimental pavement structure and theoretical predictions derived from present-day structural analysis techniques based on linear elastic and nonlinear finite-element procedures were established. The results showed clearly the necessity for taking into account the stress nonlinearity of the resilient modulus of the granular crushed-rock material in the base. On typical South African pavements it was shown that fatigue failure of bituminous surfaces occurs mainly under conditions of controlled strain. Therefore, proposals are made to modify current pavement design methods, including the more complex methods that take account of environmental conditions, traffic, material properties, and surface thicknesses. The proposals enable these methods to be applied to the design of thin (less than 50 mm) and intermediate surfaces and with the use of a computer program to predict the initiation of fatigue and the rate of deterioration of the surface stiffness caused by the propagation of cracks.

Thin Bituminous Surfaces: Their Fatigue Behavior and Prediction

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The cracking of the bituminous layer due to flexural fatigue has become recognized in recent years as a major mechanism of failure in what would otherwise be considered a well-designed pavement. The extent of that form of cracking in South African pavements is widespread (1), and this paper is concerned with the study of the main factors contributing to fatigue failure and the propagation of cracks in existing surfaces.

The pavement network of the Republic of South Africa consists of an estimated 13,000 km of roads in urban areas and 36,000 km of "black-top" rural roads. A large proportion of the 10,000-km national road network has reached the stage of requiring major improvement if it is to continue to provide adequately for present and future traffic volumes (2). Although increased use is being made of bound bases and thicker surfaces on a few of the most heavily trafficked routes, a large proportion of those routes will be improved or constructed with premixed surfaces less than 100 mm thick over unbound or stabilized aggregate base layers. Surveys of the thicknesses of bituminous surface and base specifications have indicated that there is a marked difference in South African practice compared with that of other developed countries (3) where bound bases with thick surfaces are usual. For those reasons, the main emphasis in this paper is the description of the behavior of pavements with bituminous surfaces 100 mm thick and less.

Pavement engineers have recognized that the different forms of crack patterns reveal the mechanisms of particular types of distress. That recognition can lead to the application of effective remedial action and also to the prevention of crack-inducing conditions. Conventionally, cracks can be divided into 2 groups: those due to traffic and those not due to traffic. In general, no one factor can be isolated as the sole contributor to cracking not associated with traffic because several interrelated factors are probably at work simultaneously (4).

Failures from the repeated application of loads to the pavement surface have been recognized by Hveem and Carmany (5) and have been referred to by Porter (6) and Dehlen (7). Much of that work indicates that cracking due to fatigue is the primary cause of alligator cracking. Alligator cracking in pavements, however, indicates that fatigue of the bituminous mixture is not necessarily the primary cause of the initial failure but that it is a contributory factor to the propagation of cracks in the pavements.

At the present time, inadequate knowledge exists on the intensity and location of the various forms of cracking on South African pavements on a national scale. A necessary prerequisite of a survey of that type is a rapid yet adequate method of observation, recognition, and recording of the crack characteristics. Such a method is one adopted from the work of Dehlen (8) and Williams (9).

Approximately 480 km of rural pavements on the Witwatersrand complex east of Johannesburg and 220 km of urban through-roads in 9 towns were inspected for cracking. No attempt was made in that survey to relate cracking to causative factors such as age, traffic, or pavement type. Of the 700 km of pavement covered, approximately 25 percent exhibited some form of cracking. Alligator cracking covered 19 percent of the total distance, and block, transverse, and longitudinal cracking individually covered less than 4 percent.

Those statistics clearly demonstrate the extent of the problem in this area (the Witwatersrand) and the potential problems that can arise, or may well have arisen, in all industrialized areas. Therefore, the study described above and the proposed solutions for preventing cracking must play a prominent part in the design and active prevention of this phenomenon so prevalent in South African pavements.

STIFFNESS OF BITUMINOUS MIXTURES

Stiffness of a bituminous mixture has been defined in a number of ways but is usually taken to be the ratio of an applied stress to the resultant strain and is a function of temperature T and loading time t. An apparatus (Figs. 1 and 2) was developed in 1966 (1) to measure stiffness and fatigue properties of bituminous materials.

When the nature of the loading stress is impulsive, a peak stress and peak strain can be defined; the ratio of those peak values is taken as the measure of stiffness. The stiffness thus defined is used in this paper and is termed peak stiffness.

Typical peak stiffness results as a function of temperature are shown in Figure 3, where a comparison is made between peak stiffness of laboratory specimens and field specimens for asphaltic-concrete and gap-graded mixtures. The fundamental frequency of the stress impulse, used in the test, is defined as the frequency of a continuous sinusoidal wave of which ½ cycle fits closest to the stress impulse.

FATIGUE PROPERTIES OF BITUMINOUS MIXTURES

An aspect of South African pavement structures is that the environmental and traffic conditions are such that surfaces of most of the major routes have been and will continue to be constructed or maintained at less than 100 mm in thickness. In addition, the average pavement temperatures are generally high so that bituminous mixtures operate over a low stiffness range. Those factors lead to the hypothesis that fatigue failure occurs predominantly under conditions of controlled strain as opposed to conditions of controlled stress. The necessity of establishing the applicability of those 2 conditions of loading stems from the laboratory observation that, to obtain an optimum

Figure 1. Apparatus for measuring fatigue and stiffness.

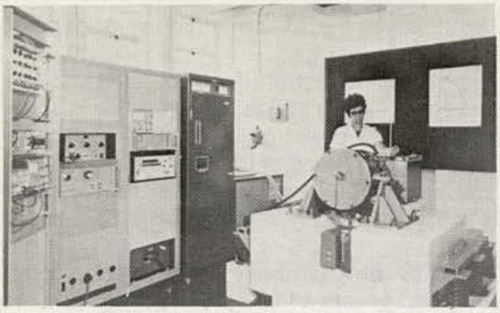


Figure 2. Loading of trapezoidal-shaped specimen.

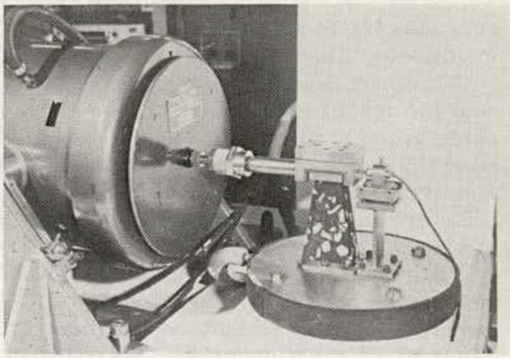


Figure 3. Laboratory and field peak stiffness values for asphaltic-concrete and gap-graded mixtures.

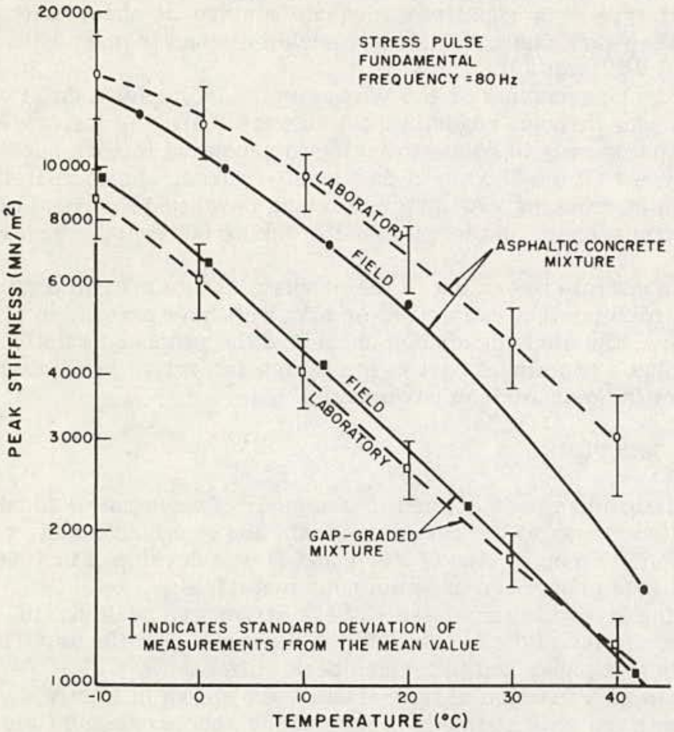
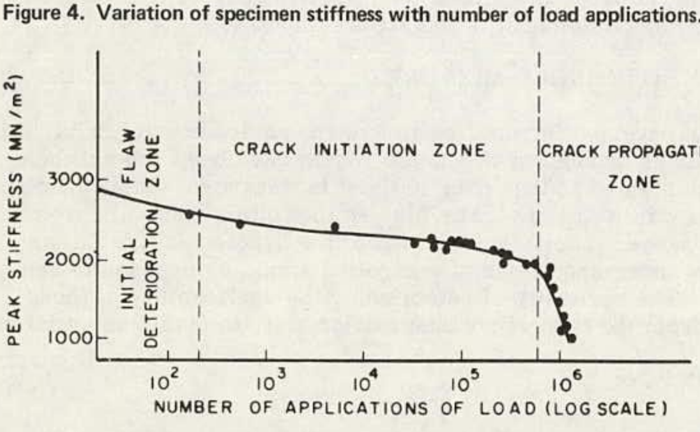


Figure 4. Variation of specimen stiffness with number of load applications.



fatigue life from a particular bituminous mixture, the required mixture constituents and temperature conditions are in general opposite for the 2 conditions of loading.

The emphasis in this section will therefore be mainly confined to aspects relating to the conditions that exist when fatigue failure occurs under conditions of controlled strain in the surface layer. Controlled stress conditions have been adequately reviewed and discussed by a number of authors, including Deacon (11), Pell (12), Kasianchuk (13), Epps (14), and Monismith (10).

Laboratory results are usually presented as a plot of fracture life against the applied stress or strain on logarithmic scales. At a particular mixture stiffness, the mean fracture life can be represented by a straight line; that relationship (15) is expressed by

$$N = K\left(\frac{1}{\epsilon}\right)^n \tag{1}$$

where N is the fracture life, ϵ is the initial strain, and K and n are constants depending on the characteristics of the mixture. Pell (15) indicated that K is of the order 10^{-16} and n varies from 5 to 6 for sand-sheet mixtures; Epps (14) indicated ranges of K from 10^{-6} to 10^{-16} and values of n from 2.8 to 5.0. That formulation for the fracture life of bituminous mixtures has been adopted as a reasonable means of expressing the results of fatigue tests and is suitable for both modes of loading (16).

For laboratory tests of the controlled strain type, the criterion of failure is not obvious, and a service life is usually defined as the accumulated number of applications necessary to cause a particular degree of failure. A comparison of the results of replicate specimens tested under controlled stress and controlled strain conditions indicated that comparable results would be obtained when specimens tested under controlled strain conditions are extremely stiff (i.e., at a low temperature), but at higher temperatures the controlled strain test results indicate considerably longer service lives than those given by controlled stress tests.

The difference between the results of the 2 modes of testing has also been explained in terms of crack propagation through the mixture. In controlled stress tests, initiation of a crack results in a reduction of the stiffness of the specimen and a subsequent increase in applied energy so that the crack is propagated almost instantaneously. In controlled strain tests, however, a considerable length of time is necessary to propagate a crack or cracks sufficiently to reach an arbitrary state when the specimen is considered to have failed or reached its service life.

Fatigue Testing Apparatus and Specimen Shapes

A more comprehensive description of the fatigue and stiffness test apparatus (Figs. 1 and 2) is given in another report (1). Aspects pertaining specifically to fatigue testing include the capability of monitoring the specimen stiffness continuously throughout the test. The feedback of either the applied stress or the resultant strain to the system input allows for the automatic maintenance of conditions of controlled stress or controlled strain. A variety of specimen shapes and sizes can be tested in vertical or cantilevered excitation, but to date most of the work has been done on cantilevered beam and trapezoidal specimens in flexure.

The conversion of the complex ratio of the applied force to the resultant displacement obtained from calibration of the electronic apparatus (17) into equivalent stiffnesses as well as the determination of the stress and strain conditions pertaining to rectangular and trapezoidal specimens is given in an earlier report (1).

Analyses made on a trapezoidal sample indicate that (a) the vertical stress and strain distributions are uniform over the lower half of the specimen, permitting a greater accuracy of prediction of these quantities than is the case when there is a rapidly varying stress-strain field in the sample, and (b) under repeated loading, the cracking of the bituminous mixtures takes place within the mixture and not at the joints to the metal specimen holders.

Type and Frequency of Loading

Deacon (11), among others, has shown that the type, frequency, and rate of application of stress are significant. A number of researchers use sinusoidal excitation methods; that technique was used in the preliminary laboratory testing programs (18), but the results revealed a number of distinct disadvantages that could be overcome by the use of the repeated application of impulsive forces. The advantages of this method are as follows:

1. The actual strain conditions occurring in a pavement surfacing, which are impulsive and half sinusoidal in nature as shown by the practical measurements (19), can be better simulated even though exact duplication is impracticable;
2. The stiffness frequency spectrum tests can be executed simultaneously with the fatigue tests by techniques similar to those outlined by Szendrei and Freeme (20, 21) but modified for this specific application; and
3. The effects of fatigue, using stresses with a combination of a number of different frequencies, can be observed.

Methods of Testing and Results

Half-sine wave impulses were used for testing the specimens. The repetition rate was chosen to be 10 times the fundamental pulse time, resulting in the application of 5 pulses per second or alternatively 1.8×10^4 repetitions per hour.

The peak stiffness was used to monitor the degradation of the specimen with number of load applications. A continuous record of the peak stresses and strain values was made on ultraviolet paper for the complete duration of the fatigue test.

A typical representation of reduction in peak stiffness with number of applications of load in the controlled strain test of the gap-graded specimen at 20 C is shown in Figure 4. The reduction in stiffness with number of applications of load is symptomatic of the deterioration of the specimen; however, the processes that give rise to this condition are not derived from one source. They are divided into 3 zones: initial rapid reduction in stiffness, crack initiation, and crack propagation.

At the commencement of the test, isolated structural flaws or, more precisely, domains of high stress concentration may exist that are sufficient to initiate the fracture or to reorient small areas of the material and result in a reduction of the stiffness. The total energy expended within the specimen is, however, insufficient to overcome the energy balance necessary to sustain that trend, and the history of the specimen enters the second phase.

Uncertainty exists in describing the actual mechanics of deterioration in the crack-initiation zone; rupture or reorientation of chemical bonds may be a cause. However, in spite of the obscurity on that aspect, this zone remains the most widely reported statistic on fatigue life. The significance of the crack-propagation zone has been, in the opinion of the authors, considerably underrated in relation to the conditions that exist in actual pavements.

The definition used of service life delineates the quotient of crack-propagation time included. For example, 3 definitions of service life are shown in Figure 5. The first designated $(N_s)_\epsilon^x$ is defined as the number of applications of load under controlled strain conditions, where the 2 lines extrapolated from the crack-initiation and crack-propagation zones intersect. The others, $(N_s)_\epsilon^{10}$ and $(N_s)_\epsilon^{50}$, are the number of applications required to reduce the initial stiffness S_1 to 10 percent and 50 percent of S_1 respectively. [For convenience, $(N_s)_\epsilon^x$, $(N_s)_\epsilon^{10}$, and $(N_s)_\epsilon^{50}$ will be symbolized by the connotation N(x), N(10), and N(50) respectively, the additional symbols of s, implying service life, and ϵ, implying controlled strain conditions, being understood. Obviously, N(50) contains information that includes a greater proportion of the crack-propagation time than the other two.]

Two further quantities are shown in Figure 5: the rates of decrease in stiffness with the logarithmic number of repetitions for both the crack-initiation zone R_i and the crack-propagation zone R_p. Those quantities are related to the stiffness S and the number of repetitions N by the general formulation

$$S = R \log N + S_1 \tag{2}$$

where S_1 is the stiffness at 1 repetition of load (i.e., N = 1).

It is recommended that the service life should be defined by N(x) for several reasons.

1. The strain versus service life can be conventionally plotted where N(x) is understood to imply the number of applications of load to cause crack initiation, and Eq. 1 (15) becomes applicable.
2. With the knowledge of R_i and S_1 available, the stiffness at the onset of crack initiation can be computed.
3. Further, if R_p is then available, the stiffness after any number of applications in the crack-propagation zone may be computed; or, conversely, the fatigue life may be defined at any arbitrary state of deterioration of the stiffness.

The results of the fatigue tests show that the rate of crack propagation R_p is markedly dependent on temperature as shown in Figure 6. The value of R_p for 0 C is of the order of 20 to 100 times its value at 40 C for the gap-graded and asphaltic-concrete mixtures respectively. The dependence of R_i on temperature is, however, insignificant, as Figure 4 also shows.

The experimental approach adopted to obtain the results on the bituminous surfacings was specifically chosen to minimize the time required to carry out the time-consuming fatigue tests. The aim was to establish the values of the K and n coefficients (Eq. 1) at 20 C with reasonable accuracy (as given by the coefficient of variation of the points from the best straight line) and then to make the assumption that the n coefficient does not vary significantly with temperature. That assumption, which seems reasonable on the basis of the results of the literature review (1), enables the tests at other temperatures to be made at one strain level only to establish the value of the K coefficient.

Figure 7 shows the dependence of service life N(x) on strain level at 20 C. The gap-graded mixture has a significantly greater service life than the asphaltic-concrete mixture under equivalent conditions. An alternative definition of service life, such as N(50), would have resulted in a significantly different graph, mainly as a result of R_p being dependent on temperature. N(x), the service life used, should be interpreted as the number of repetitions required to cause crack initiation.

The n values obtained with this apparatus were of the order of 5 and 6 and tend to agree with values obtained by Pell (15) rather than those reported by Epps (14).

The dependence of these service life curves on temperature can best be demonstrated by the variation in the K value in Pell's equation with temperature (K can be considered to be the service life at a strain level of 1 μs). That is shown in Figure 8a where the logarithm of K has been plotted as a function of temperature. The trends obtained here once again agree with those obtained by Pell.

In general, changes in the fatigue life of a bituminous mixture can be attributed to changes in the stiffness of the mixture. For example, under controlled strain conditions, factors that tend to reduce the stiffness of the mixture generally increase the fatigue or service life; under controlled stress conditions, the reverse applies. For that reason, the value of the K parameter, which in this case is a direct measure of the service life, has been plotted as a function of the peak stiffness. Figure 8b clearly shows that the difference between the gap-graded and asphaltic-concrete mixtures cannot be attributed solely to the differences in their peak stiffness. That result agrees with the observations made by Epps (14) on the differences between the California continuously graded and British BS-594 gap-graded mixtures.

However, it has been shown (1) that the bulk modulus measured by means of the ultrasonic concrete tester is primarily dependent on the voids in the mixture and the temperature of the mixture, characteristics that also apply to the service or fatigue life. For that reason, the value of the K parameter at different temperatures is shown in Figure 8c as a function of the bulk moduli of the 2 mixtures. In such a test, the marked difference between the 2 mixtures is eliminated, and an almost unique relation results. That relation has not been shown to be valid for other mixtures, but the important implication of this finding is that the potential exists for being able to determine the fatigue coefficient K simply by making a measurement of the bulk modulus. The

Figure 5. Definitions of service life.

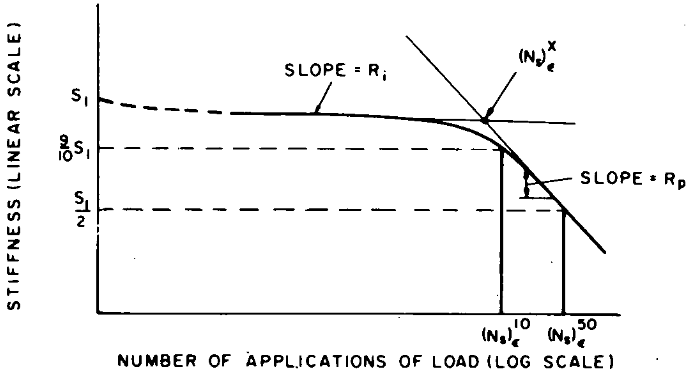


Figure 6. Variation of R_i and R_p with temperature.

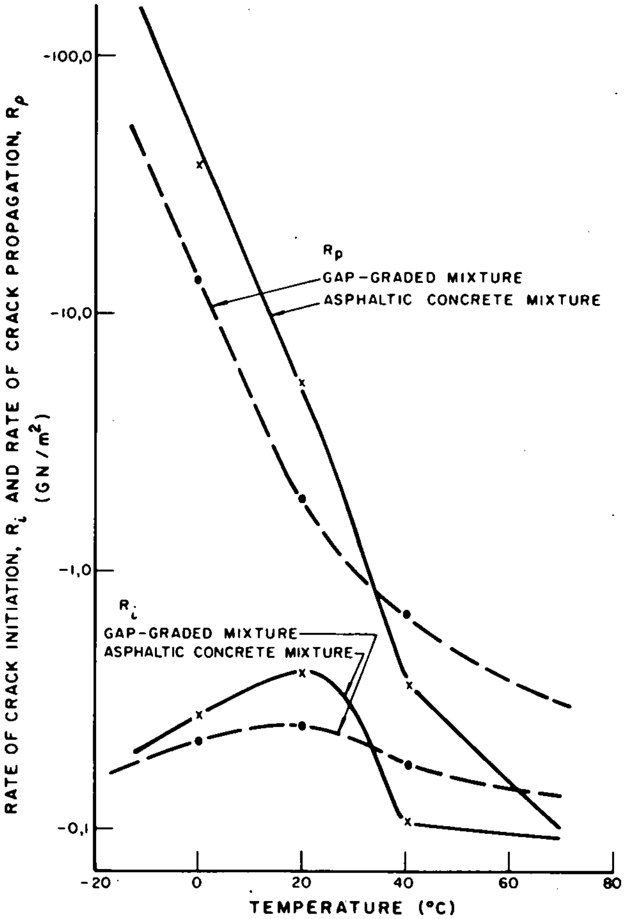


Figure 7. Service life strain for experimental pavement surface.

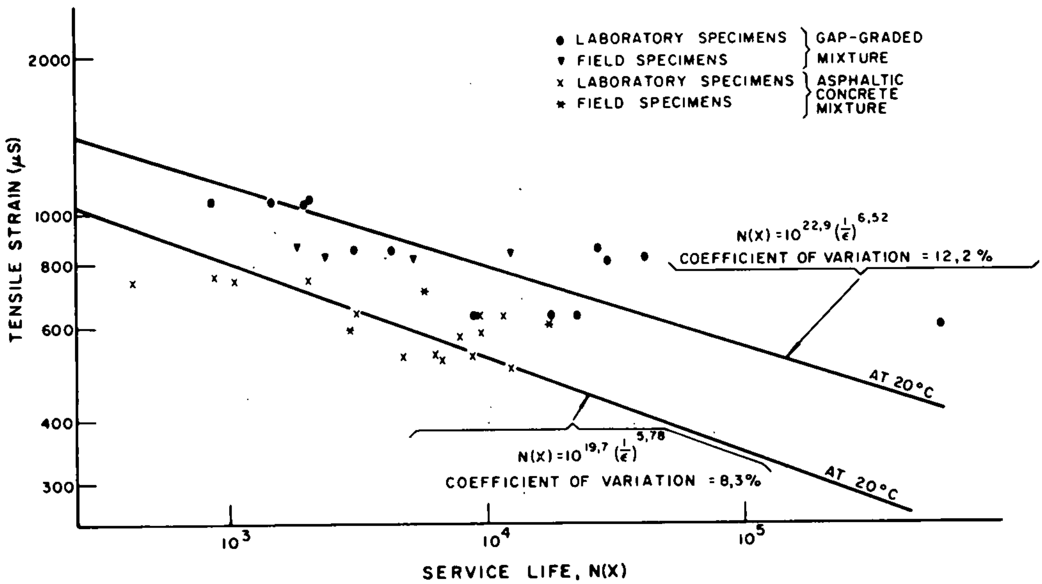
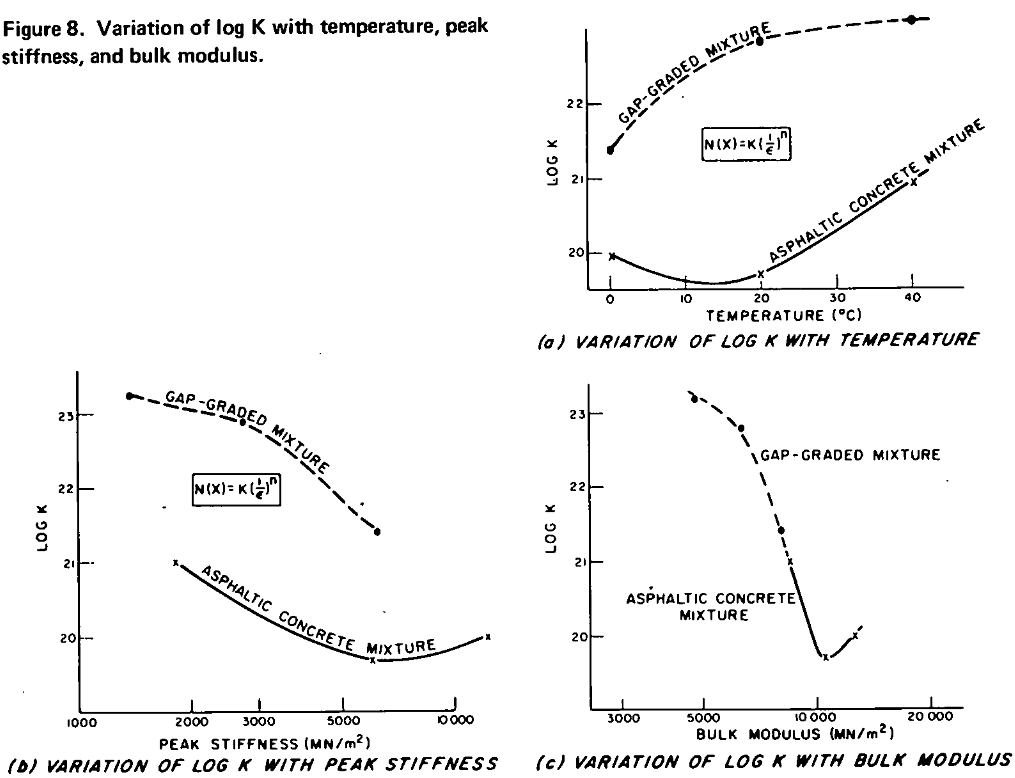


Figure 8. Variation of log K with temperature, peak stiffness, and bulk modulus.



nondestructive determination of the bulk modulus can be made with the ultrasonic concrete tester, a commercially available instrument, in a matter of minutes on specimens obtained by using one of the compaction machines, such as the Marshall or gyratory compactor.

MEASUREMENT AND PREDICTION OF DEFLECTIONS AND STRAINS IN ASPHALT PAVEMENTS

Advanced methods of pavement design are currently being studied and developed throughout the world. In those procedures, attempts are made to account for the characteristic effects of fatigue in the bituminous layers at the top of the pavement structure in order to eliminate that type of distress. Also, it is necessary to establish the applicability of existing techniques for predicting deflection, stress, and strain within the pavement structure and to correlate them with the actual loading and environmental conditions experienced in practice.

A detailed study was undertaken of a heavily trafficked experimental asphalt pavement in which both linear-elastic and non-linear-elastic finite-element analytical techniques were applied (19). The analytical procedures used were examined on the basis of a comparison between the measured and predicted behavior of the pavement layers; where significant differences were encountered, the material properties of the pavement layers were reassessed more realistically and incorporated in the analytical procedures.

That study was concerned, in particular, with the characterization of the properties of individual layers of the pavement. They included 2 types of bituminous surfaces, a granular crushed-rock base, a weakly lime-stabilized layer, and a selected subgrade material. In situ pavement test techniques, such as the CBR and wave-propagation methods, were combined with information obtained from the laboratory investigations to establish the most realistic elastic moduli for each pavement component. In the case of the granular crushed rock, the necessity for taking the stress nonlinearity of the modulus into account was clearly demonstrated.

Effects of surface type, temperature, and thickness on the maximum tensile strain at the surface-base interface were investigated both practically and theoretically on different sections of the experimental pavement. Finally, predictions were made of the most probable values of maximum tensile strain in the different surfaces over the wide range of environmental, traffic, and structural conditions likely to exist at the experimental pavement.

DESCRIPTION OF THE EXPERIMENTAL ASPHALT PAVEMENT

Experimental sections of the asphalt pavement were constructed late in 1968 on Special Road 12/2, a heavily trafficked route in the Highveld climatic area of Transvaal, South Africa, and were opened to traffic in July 1969.

Five sections with gap-graded surfaces and 5 sections with continuously graded asphaltic-concrete surfaces, each approximately 50 meters long and 25, 38, 50, 78, and 100 mm thick, were constructed on the pavement structure as shown in Figure 9. The selected subgrade layer, which varied in thickness from 1.5 to 2.0 meters, formed an embankment on either side of the bridge that was located within the experimental length. The in situ subgrade was a moist clay approximately 1.2 to 2.1 meters thick and provided a resilient material.

Strain meters, of the type developed by the Great Britain Transport and Road Research Laboratory but slightly modified (22), were installed in 6 of the 10 experimental sections. Three strain meters to measure horizontal longitudinal strains were installed at each site at the surface-base interface on a longitudinal line in the outer wheelpath of the westbound traffic lane. Thermocouples to measure temperatures were also installed at the surface-base interface close to the strain meters. For both sections with 100-mm thick surfaces, additional thermocouples were placed at the midpoint, i.e., 50 mm below the top of the surface (Fig. 9).

At one site (the section with a 100-mm gap-graded surface), 4 reference rods incorporating linear variable differential transformers were installed to determine variation of elastic deflection with depth under moving traffic loads. The lower end of each rod was located at 4 different depths below the top of the surface. The deflection value measured was the difference in position between the top of the surface and the end of each rod. Only the elastic (short-term) component of the deflection was taken into account.

ENVIRONMENTAL AND TRAFFIC CONDITIONS AT THE EXPERIMENTAL PAVEMENT

Thermal Environment

The part played by diurnal and annual variations in the temperature of the bituminous surface should be recognized as being of paramount importance in identifying the most critical conditions for the initiation and propagation of cracks. However, defining the most critical conditions does not give a complete answer, and the use of those conditions alone, which exist for only a portion of the design life, will probably result in the overdesign of the surface. Establishing the extent to which that is likely to occur is essential and can best be done by investigating the damage that occurs within specific temperature regimes during the life of a pavement. Initially, however, that requires either the recording of the temperature within the pavement surface or the prediction of the likely temperature distributions from weather data recorded in the vicinity of the pavement.

Williamson (23) has reviewed a number of aspects of the accuracy of 4 basic approaches to the problem of temperature prediction in pavements: The Barber (24) solution, which is a formal mathematical equation governing one-dimensional transient-heat conduction in a semi-infinite medium; the finite-element approach as reported by Wilson (25); a finite-difference solution to the heat flow equation, which involves recording data at preselected intervals, developed by Schenck (26); and a regression analysis method developed by Southgate and Deen (27).

The Wilson and the Southgate and Deen methods showed no advantages over those of Barber and Schenck, and the Schenck method appeared to be the most useful for accuracy of prediction, especially at the lower temperatures. The Barber method is one of the most useful temperature-prediction methods but becomes inaccurate at low temperatures.

Kasianchuk (13) investigated the influence of the variation in the constituents of bituminous mixtures within the ranges derived from the summary of available data presented by Finn (28) and concluded that the use of typical values would prove acceptable for design purposes. That is particularly valid when consideration is given to the greater influence likely to result from the meteorological variables of solar radiation and wind velocity.

Climatological Data for the Experimental Pavement

In the previous section, the importance of obtaining the climatological variables at the location of the experimental facility was indicated. Attempts to obtain realistic data led to the formulation of 2 approaches: (a) the assessment of the average conditions during a number of years, primarily obtained from the general survey as reported by Schultz (29), and (b) the reduction in the available data for the period after completion of the pavement, that is, after the beginning of 1969.

Information on the actual values of the climatological data in the vicinity of the experimental pavement has been accumulated from December 1968 to December 1970 from data supplied by the Weather Bureau at Pretoria. That information is given in another report (1) and includes the minimum air temperatures and the wind velocities taken at Jan Smuts International Airport. Mean values and standard deviations of those variables were computed on a monthly basis.

Excellent comparisons between the information averages during a number of years and the information in the 1969-1970 period were obtained and implied that the average data predicted for a given year could be used with reasonable confidence to predict average trends. However, details of particular events, which may be of importance, will be lost in this process. For example, a cold spell may induce acceleration of the fatigue process in thin surfacings. In that case, actual data may be of more importance, or alternatively the use of the concepts for the prediction of the duration and intensity of particular spells may prove of greater value in those predictions.

Traffic Measurements and Predictions

The existing experimental pavement was completed in December 1968 and officially opened to traffic in June 1969. Some traffic did, however, use the facility prior to June 1969. Up to the present time, 2 types of traffic analyses have been made on the pavement: traffic counts on a fairly routine basis from April 27, 1970, and axle weight distributions, which were measured by means of a portable axle weight analyzer as described by Freeme (30). Three sets of weekly measurements were made with the axle weight analyzer during August 1970, February 1971, and March 1971.

Variations in the traffic counts in 1 direction were assumed to be linear during the period from December 1968 for approximately 1,000 days, after which a more normal annual growth rate of some 6 to 8 percent could be expected. The variations in the higher axle groups are shown in Figures 10 and 11. Generally, similar distributions are reflected for the 3 sampling periods, and a mean distribution was accepted as being representative of the traffic distribution.

PREDICTION OF FATIGUE LIFE OF ASPHALT PAVEMENTS

The fatigue prediction procedures available at the present time are predominantly applicable to the prediction of the initiation of fatigue in thick bituminous layers (approximately thicker than 100 mm). The procedures have been modified so that they can be applied in the design of thin (less than 50 mm) and intermediate surfaces. The applicability of those methods was assessed by the use of the information reported in earlier chapters to estimate the progressive development of fatigue in the different surface thicknesses of the experimental pavement.

Discussion on the Fatigue Prediction Procedures

A primary objective of the procedures for the prediction of fatigue in bituminous layers should be to assess the contribution that is made by each variable to the fatigue subsystem and that culminates in improved design techniques. Unfortunately, improvements in a particular aspect often result in more complex data acquisition programs. For example, the information required by the Shell 1963 design method (31) can be obtained fairly easily by most engineers and the design procedure completed in a matter of hours. The computer program employed in the University of California procedure (13), however, requires data that may take months to accumulate. A compromise between utility and possible improvement in design accuracy is the procedure advanced by Brown and Pell (32).

Identification of the Most Critical Conditions for Fatigue

In the design methods advanced by the Shell Petroleum Company (31) and Nottingham University (32), certain critical criteria are given that should not be exceeded in the pavement structure. Procedures of that type are used in current design techniques such as those of bituminous mixtures where minimum stability and a range of flow values number among the criteria specified to achieve adequate in-service stability. Because temperature is one of the most critical environmental variables in determining the behavior of bituminous mixtures, the laboratory measurement of those stability values is made at a temperature at which the mixture is most vulnerable (usually 60 C for warm and 45 C for cold climatic areas).

However, in the specification of the criteria for the prevention of failures due to fatigue, confusion exists in the description of the most critical temperature conditions.

Figure 9. Longitudinal section of experimental asphalt pavement.

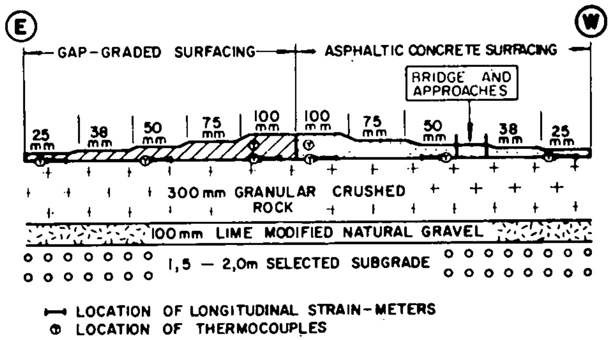


Figure 10. Load distribution of average daily count on experimental pavement.

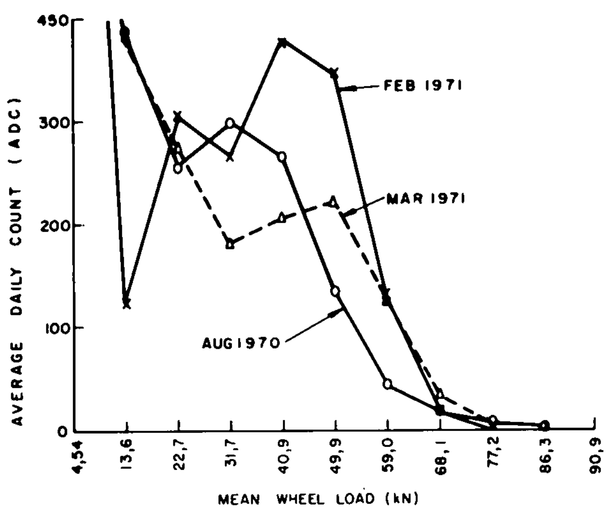
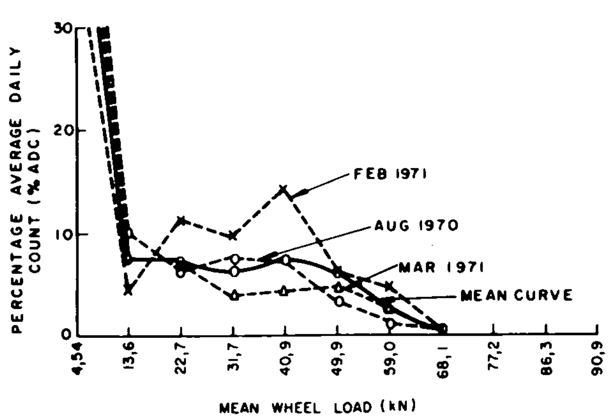


Figure 11. Load distribution of percentage of average daily count on experimental pavement.



Some clarification may be obtained by considering the results from the structural analysis procedures in combination with the fatigue testing results. Two cases will be considered: asphalt pavement with a thin bituminous surface and pavement with a thick bituminous layer such as a combination of base and surface layers. Attempts will be made to define the criteria and the conditions under which those can be used in the prediction of fatigue failures.

Pavement With Thin Bituminous Surface

In asphalt pavement structures with thin surfaces (approximately less than 50 mm thick), conditions exist such that

1. Ideally, the variation in maximum tensile strain at the surface-base interface is independent of the surface temperature (or modulus) (it is assumed that although the influence of the variations in the base and lower layers changes the absolute magnitude of the strain it does not affect its dependence on temperature); and
2. Controlled strain fatigue test results are applicable, and that implies that an increase in temperature will result in an increase in fatigue life.

Those factors lead to the hypothesis that for those pavement structures with thin surfaces (that is, less than 50 mm) the most critical temperature conditions that will produce a minimum fatigue life occur at low temperatures. Hence, at that stage, for thin surfaces, it is recommended that the critical values of maximum tensile strain be specified at a low temperature. Further, as concluded by Pell, the indications are that at very low temperatures (less than 0 C) the fatigue life becomes independent of temperature, producing results comparable to those produced under controlled stress conditions.

Hence, one may derive maximum permissible values of tensile strain, such as those obtained by Epps (14) and Pell (32), from the considerable data on controlled stress tests available in the literature by assuming the data to be equivalent to those that would be obtained under constant strain conditions at low temperatures.

Variations in the constituents of the mixture will influence the maximum permissible tensile strains; for example, an increase in void content will reduce the magnitude of the permissible strains. At present, however, it is recommended that the values given in Table 1 be used, for they will lead to a conservative design for the prevention of fatigue distress provided the most appropriate mixture is chosen to obtain representative values for the design under consideration.

Pavement With Thick Bituminous-Bound Layer

In pavements having a thick bitumen-bound layer at the top of the construction (for example, a surface plus base), the conditions are such that

1. The maximum tensile strain at the bottom of the bituminous layer must decrease with an increase in layer modulus or equivalently with a decrease in layer temperature; and
2. The data resulting from controlled stress laboratory tests on fatigue are applicable, and that implies that the maximum tensile strain for obtaining a particular fatigue life is virtually independent of temperature.

If those 2 factors are combined, the fatigue life of thick bituminous-bound layers decreases with an increase in temperature—precisely the opposite trends to those shown by a pavement with a thin surface.

Hence, from those considerations, the most critical conditions, as far as fatigue life in a thick bitumen-bound layer is concerned, will occur when the temperature of the bitumen-bound layer is at a maximum.

The maximum allowable tensile strain values given in Table 1 for various mixtures still apply in the case of the thick bituminous layers. The main difference is in the application of those values. For thick bituminous layers, the minimum moduli of the bitumen layers should be used in the structural analysis; for a thin surface, the stiffness of the surface is not critical, and average values can be used in the structural analysis.

Pavement With Intermediate Surface

For pavements that have intermediate bituminous layers, a more complex interrelation exists in that both the maximum tensile strain at the bottom of the layer and the fatigue life are dependent on temperature. Further, the fatigue data from controlled stress or controlled strain laboratory tests are not strictly applicable but rather data from some combination of the 2 tests. That implies that, although the values given in Table 1 could be used, each individual pavement structure must be analyzed on an individual basis over a wide range of temperature to determine the most critical conditions.

Use of Axle Weight Equivalency Factors in Fatigue Prediction

It is doubtful whether equivalency factors, such as those obtained from the AASHO Road Test (33), can be applied in the conversion of a spectrum of axle loads to an equivalent axle load of single value and that information can be used to predict the fatigue behavior of bituminous surfaces. That procedure, which is employed in both the Shell and University of Nottingham design methods, results in the high axle loads being primarily responsible for the fatigue damage to the bituminous layer. For example, for a pavement with a structural number of 4 and a serviceability index of 2.0, that would result in 1 axle load of 82-kN being equivalent to 5,000 axle loads of 4.5-kN (34). The assumption here is that heavier axle loads produce larger tensile strains in the bituminous layer than lighter axle loads. That increase, magnified by the radical dependence of the fatigue life on the level of tensile strain, could conceivably produce values that would justify the equivalency conversion. However, if the maximum tensile strain decreases with an increase of axle load under particular circumstances, then clearly erroneous values of the fatigue life will be predicted.

Grant (35) indicated that this situation could occur when the tensile strains under thin surfaces are considered. A theoretical analysis of 3 pavement structures was investigated by the use of a linear-elastic multilayer computer program. Two of those pavements had surfaces 25 mm thick, representative of typical South African pavement structures, and the third had a 100-mm surface, representative of the type used by the Shell Company in the development of its 1963 design charts.

For the systems with 25-mm thick surfaces, the radial strain located directly under the wheel load decreased with an increase in wheel load, becoming compressive at high loads. For the system with the 100-mm surface, an increase in tensile radial strain with increasing wheel load was observed.

The maximum tensile strain in the thin surface is primarily dependent on the tire-contact pressure; higher tire-contact pressures result in a marked increase in the tensile strain. Because the heavier loads (trucks) generally have higher tire pressures (and higher tire-contact pressures) than lighter axle loads (cars), the marked decrease in tensile strain associated with the heavier axles is to some extent compensated.

On pavements having thick bituminous layers, however, the net effect of wheel load and tire contact pressure is that the maximum tensile radial strain increases significantly with an increase of wheel load for the experimental pavement indicating that probably axle weight equivalency factors can be applied to account for the effects of fatigue initiation in thick bituminous layers.

Based on those observations relative to the difference in dependence of thin and thick bituminous layers to wheel loads and tire contact pressures, it is postulated that in the prediction of fatigue life using procedures similar to those of the University of Nottingham it is essential to distinguish between pavements with different surface thicknesses.

1. For pavements with thick bituminous layers (greater than 150 mm), equivalency factors, such as the AASHO equivalency factors, can be used to reduce the axle load distributions to an equivalent value;
2. For pavements with thin surfaces (less than 50 mm), it is more correct to use in the elementary fatigue-prediction procedures the actual number of axles, irrespective of wheel load, to account for the effects of repetitions of load; and
3. In all cases, it is more accurate to determine the actual influences of wheel load

and tire contact pressure by using authenticated structural analysis procedures to account for the effect of repetitions of load by making use of a cumulative damage hypothesis.

PREDICTION OF FATIGUE INITIATION AND CRACK-PROPAGATION LIFE IN THE EXPERIMENTAL PAVEMENT

A computer program was developed to validate the observations made in the previous section and to predict the onset and rate of development of fatigue in the surfaces of the experimental pavement.

National Institute for Road Research Fatigue Prediction Program

The computer program developed is presented in more detail in another report (1). It combines the most important research findings and was originally based on the computer program reported by Kasianchuk (13). However, some significant improvements have been made to the original program.

1. The program computes not only the onset of fatigue (crack initiation) but also the rate of deterioration in the stiffness of the surface in both the crack-initiation and crack-propagation zones.
2. The program can take account of the decrease in stiffness of the surface due to the repeated application of load, a fact that is extremely important in the prediction and measurement of the life of surfaces less than 100 mm thick.
3. It uses information on traffic obtained from the portable axle weight analyzer system developed at the National Institute for Road Research (NIRR).
4. Results of either controlled stress or controlled strain fatigue tests can be used.
5. Although in the original program developed by Kasianchuk only monthly averages of the traffic and environmental data could be used, the NIRR program is able to use monthly, weekly, or daily averages. That permits the investigation of the significance of factors such as cold or warm weather and the determination of diurnal variations and, if necessary, permits them to be taken into account. If those variations are found to be of importance, then the average frequency of occurrence of those events can be included in the computer program.
6. The program also provides information on and the opportunity to study contributions to the Miner criterion $\Sigma\{n/[N(x)]\}$. That allows the investigation of the contribution of the individual wheel-load groups in any month or year and the contribution of all the axles in each individual month.
7. Stiffness or traffic-weighted mean stiffness of the surface computed from the climatological data can be used for actual data measured during a specified period (such as data obtained during the 1969 to 1970 period for the experimental pavement), or average values can be used for as many years as desired.

Fatigue Results of Experimental Pavement

Fatigue Initiation

Prediction of the fatigue initiation (that is, the time taken to initiate a crack at the surface-base interface) is made by using the linear summation of cycles ratio (Miner criterion). The sum of the cycle ratio was taken during consecutive months, and 2 fatigue initiation values were computed: the sum when the 90 percent confidence level reaches unity and the sum when the mean value reaches unity. The results of those predictions are given in Table 2.

The gap-graded surface sections will have significantly greater fatigue-initiation lives than the equivalently thick asphaltic-concrete surfaces. Half the number of years are required to initiate a crack in the 50-mm gap-graded surface than in both the 25-mm and 100-mm surfaces (approximately 8 years as compared to 16 years). The 50-mm asphaltic-concrete surface has the shortest fatigue initiation life and should show signs of failure early in 1972. The 25-mm surface should only begin to fail in 1973, and the 100-mm surface is not expected to reach that stage until between 1977 and 1979.

Those predictions of fatigue life were made in 1971 (1), and some confirmation of their validity was obtained in 1972 in that fatigue cracking was observed visually in the 50-mm asphaltic-concrete surface but in none of the other surfaces.

The predictions given above were obtained by using controlled strain, fatigue-test data, and that mode of loading is probably only applicable to a surface that is less than 50 mm thick. Although controlled stress results are also not applicable to 100-mm surfaces either, they do provide a lower limit for the fatigue predictions of that surface thickness. Comparison of using fatigue data on controlled stress and on controlled strain for the 100-mm asphaltic-concrete surfaces showed that the period for fatigue initiation is reduced from approximately 10.5 to 8 years. However, the 100-mm asphaltic-concrete surface still has the best fatigue life when compared to the 50-mm and the 25-mm surfaces even under those conditions.

Stiffness Reduction and Crack Propagation

It was shown that the stiffness of the bituminous mixture decreases with an increase in number of applications of load. The results have been expressed as a percentage of the original stiffness because that eliminates the effects of temperature that occur as a result of the monthly variations in surface temperature.

The percentage reduction in the surface stiffness as a result of the application of the mixed axle-load spectrum under the conditions at the experimental pavement is shown in Figure 12 for the gap-graded surface and Figure 13 for the asphaltic-concrete surface. Both figures show that an initial rapid reduction in stiffness is obtained in the first year, and thereafter the decrease in surface stiffness is small until the crack is initiated at the surface-base interface. After that stage, the stiffness of the surface decreases approximately linearly with the number of years.

It should be possible to follow the progress of fatigue by observing the percentage reduction in surface stiffness. Continual records of the surface stiffness, at say yearly intervals, should show the following trends.

1. Apart from the first year, if the percentage reduction in surface stiffness is minimal, the surface will still be in the crack-initiation zone and performing from the flexural point of view as expected. [Traffic compaction, however, may increase the bulk density of the surface, thereby increasing the stiffness during the first year (36).]
2. Should there be a more rapid linear reduction in surface stiffness, that will indicate that cracks have been initiated. A linear extrapolation will indicate whether the original stiffness of the surface has decreased to the particular percentage of the original that constitutes failure (for example, 50 percent of the maximum stiffness).

Both Figures 12 and 13 clearly show that, if a crack is initiated rapidly in a particular surface, propagation of the crack will be rapid. Conversely, if it takes a relatively long time to initiate the crack, the time for the propagation of the crack will be long. Those results assume that no deterioration in the other pavement layers, such as the base, occurs during that period.

COMPARISON OF SHELL, UNIVERSITY OF NOTTINGHAM, AND NIRR FATIGUE PREDICTION PROCEDURES

A comparison was made of the results obtained by using the NIRR method and the results obtained by using simpler methods such as the Shell (31) and University of Nottingham (32) design procedures.

Comparison With Shell Procedure

The Shell method can only be used for thick bituminous-bound layers over a granular base and a subgrade with a known CBR value. In the case of the experimental pavement, the lime-stabilized layer (with a CBR of approximately 40) constituted a problem because

Table 1. Maximum permissible tensile strains in surface layers for various mixtures.

	Maximum Allowable Tensile Strains (μs)					
Applications of Load	Dormon and Metcalf Criteria (37)	California Dense Graded[a]	California Graded to Meet Extremes[a]	California Medium Graded[a]	BS-594 Tested by Epps (18)	BS-594 Tested by Pell (36)
10^3	–	877	868	1,029	951	534
10^4	–	408	395	488	481	365
10^5	230	190	179	231	243	250
10^6	145	88	81	109	123	171
10^7	92	41	37	52	62	117
10^8	58	19	17	25	32	80

[a]Values of more than approximately 10^5 repetitions of load are extrapolated from the data of Epps (18); a linear relation is assumed between the logarithm of the strain and the logarithm of the fatigue life.

Table 2. Prediction of mean and shortest possible fatigue initiation period by various methods.

	Gap-Graded Surface		Asphaltic-Concrete Surface	
Surface Thickness (mm)	Shortest Possible Period (years)	Mean Period (years)	Shortest Possible Period (years)	Mean Period (years)
100	12.4	15.5	9.1	11.0
50	6.5	8.3	3.1	3.6
25	12.7	16.1	4.2	5.0

Table 3. Prediction of fatigue initiation period by University of Nottingham method and total traffic count.

Method	Thickness (mm)	Gap-Graded Surface (years)	Asphaltic-Concrete Surface (years)
Nottingham	100	>40	>40
	50	19	2.3
	25	>40	7.0
Total traffic	100[a]		
	50	16	4.6
	25	31	4.0

[a]Not applicable.

Figure 12. Reduction in gap-graded surface stiffness with number of years.

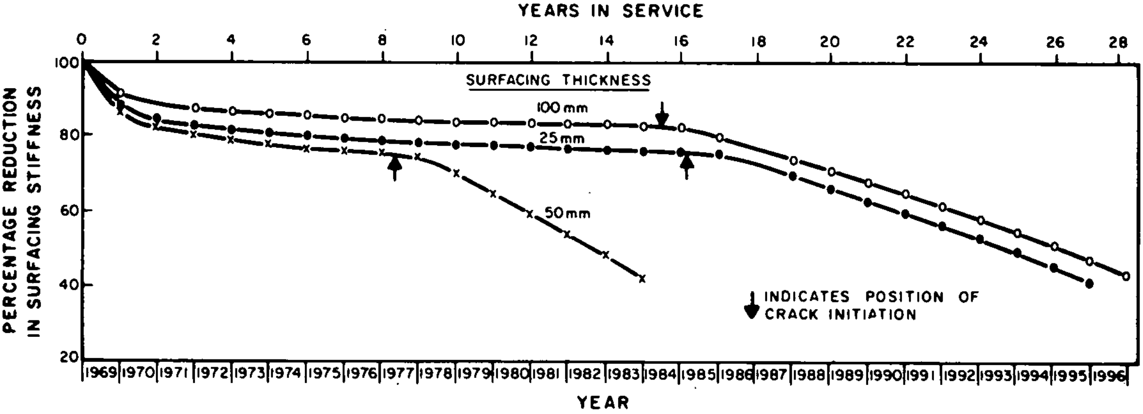
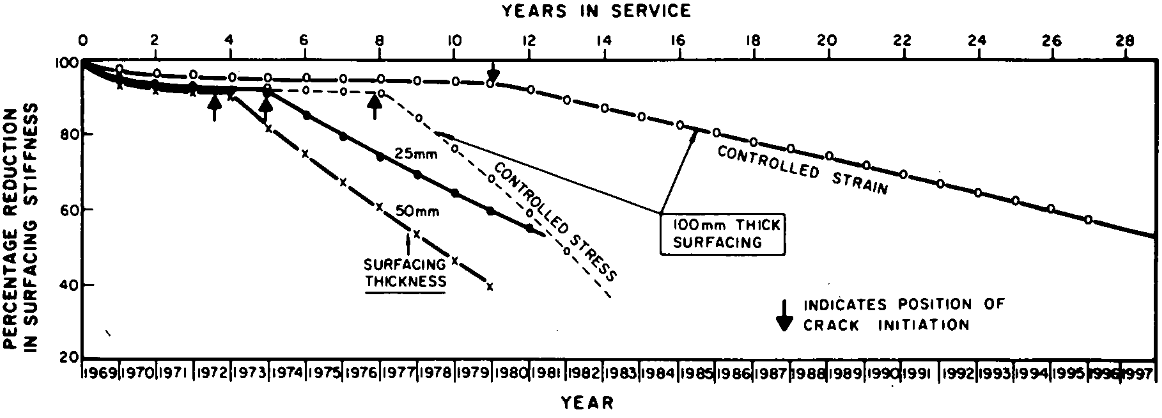


Figure 13. Reduction in asphaltic-concrete surface stiffness with number of years.



the maximum CBR specified in the Shell design charts is 17. However, that value for the CBR of the subgrade was used to predict the fatigue life of both 100-mm gap-graded and asphaltic-concrete sections to be approximately 3 years. The fatigue life of the 50-mm sections was predicted to be less than 1 year. Because those results do not take into account the significant difference between the 2 surface mixtures, they do not compare very favorably with the results given in Table 2.

Comparison With University of Nottingham Procedure

In the University of Nottingham method, the fatigue results for the specific mixtures at 20 C were used. That temperature was considered the most critical for the thin surfaces, for it is the lowest mean monthly temperature for the experimental pavement.

The traffic factor was taken into account by computing the equivalent number of legal wheel loads (dual-wheel configuration). The maximum tensile strain at the surface-base interface of the pavement was obtained at stiffnesses of 2,800 MN/m^2 for the gap-graded surfaces and 6,000 MN/m^2 for the asphaltic-concrete surfaces.

The initiation of fatigue was then predicted as being likely to occur after the number of years given in Table 3. Those values should be compared with the mean fatigue initiation period given in Table 2. The data demonstrate that this simplified method does not produce results that agree quantitatively with the more precise NIRR method. It is clear that the 50-mm surfaces are the most critical; the asphaltic-concrete surface reflects an extremely low fatigue initiation period. The interpretation to be placed on the surfaces with fatigue initiation periods greater than 40 years is that fatigue is unlikely to constitute a significant problem in those pavements.

A primary recommendation given in this paper is that for pavements having thin surfaces it is more correct to use in elementary fatigue prediction procedures the actual number of axles irrespective of wheel load to account for the repetitions of wheel load. It was also demonstrated that cars and light, single wheel-load axle groups are the predominant contributors to the linear summation of cycle ratios, and for that reason the 13.6-kN wheel-load group was chosen as being representative of all the wheel loads. The fatigue predictions taking these factors into account are also given in Table 3 and show superior predictions in absolute magnitude as is illustrated by comparison with results from the NIRR method given in Table 2. The results demonstrate that from a design point of view methods such as those used in the University of Nottingham design procedure can be used to obtain an approximate and possibly a reasonable fatigue prediction result.

PRACTICAL SIGNIFICANCE OF THE FATIGUE RESULTS

Several concepts of considerable practical significance have resulted from the consideration of the factors that contribute to the fatigue failure of bituminous surfaces. Procedures involving those concepts have been combined in a computer program that permits the prediction not only of the initiation of fatigue but also of the deterioration in surface stiffness in both the crack-initiation and crack-propagation zones. Data necessary for the prediction of the fatigue behavior of the different sections of an experimental pavement were accumulated throughout the experimental work on which this paper is based, and the results obtained support and confirm the concepts resulting from the general considerations of the properties of the material and the structural behavior of the pavement. The most important practical results of this work are discussed below.

Surface Type and Stiffness

The use of gap-graded mixtures of low stiffness in place of continuously graded asphaltic-concrete mixtures for surfaces less than 50-mm thick can result in a significant improvement in the fatigue life of the surface and its resistance to the propagation of cracks. An improvement by a factor of 2 was predicted in the fatigue-initiation and crack-propagation zones of the gap-graded surface sections of the experimental pavement, including the surfacing that is 100 mm thick. Whereas the predictions indicated that fatigue is unlikely to be a cause of failure of the pavement where the gap-graded mixtures are used, fatigue will constitute a problem where asphaltic-concrete surfaces are used. That last observation is supported by actual field observations of cracking due to fatigue in the 50-mm asphaltic-concrete sections after only 2.5 years of service.

The 2 most important reasons for the improved fatigue behavior and resistance to cracking of the gap-graded mixtures are that the stiffness, or more pertinently the bulk modulus, of the mixture is significantly lower than that of the asphaltic-concrete mixture and that the voids in the mixture are uniformly distributed, small, and not easily discernible.

Reduction in the stiffness of the mixture can be achieved in a number of ways. The most important of those from the practical point of view appears to be in the use of high penetration-grade (low viscosity) binders and the selection of open or gap-graded aggregate gradings. (The bitumen penetration grade selected should not be so high that the mixture deforms excessively under traffic loads.) A further method of reducing the stiffness of the mixture is by reducing the filler content (according to a private discussion in 1971 with staff of the Natal Roads Department, Peteirmaritzburg, South Africa).

Most Critical Surface Thickness

When the 50-mm thick surfaces are supported by granular crushed-rock bases, they are the most susceptible to early failure and rapid crack propagation. The results of the predictions on the experimental pavement indicated that the fatigue initiation period of the 50-mm surfaces is approximately half that of the 25-mm and 100-mm surfaces irrespective of surface type.

A practical result of that finding is that in the design of pavements with unbound bases care should be taken either to keep the surfacing as thin as possible or, if necessary from the point of view of reducing flexure, to incorporate a substantial thickness of bituminous-bound material so that this layer is capable of supporting the bulk of the load.

That finding is dependent on the type of base material and in particular on the ratio of the surface stiffness to the base modulus. When the surface stiffness is high in comparison with the base modulus (i.e., in the range of 10:1 to 4:1), the most critical surface thickness is of the order of 50 mm. However, if the base modulus is increased, for example, by adding stabilizing agents to the unbound base, then the most critical surface thickness also increases.

Most Critical Temperature Conditions

The temperature at which the most rapid rates of fatigue failure and crack propagation occur depends on the thickness of the surface. For thin surfaces (less than 50 mm thick), the most critical temperature conditions occur when the surface mixture is cold; but, in thick bituminous-bound layers (approximately greater than 150 mm), the most critical conditions occur when the temperature is high.

Effects of Wheel Load and Tire Pressure

In surfaces that are less than 100 mm thick, it has been clearly demonstrated both theoretically and in the field that the tire contact pressure has a far greater effect on the fatigue life of the surfaces than the wheel loads of the vehicles. In the light of the discussion above and other more detailed results of this study, the authors suggest that the following procedures be employed for dealing with the effects of mixed traffic when fatigue life of bituminous surfaces is predicted by design procedures such as the University of Nottingham method.

1. For pavements with thick surfaces (greater than 100 mm), use equivalency factors such as those obtained from the AASHO Road Test.

2. For pavements with thin surfaces (less than 50 mm), use the actual number of axles, irrespective of load.
3. For pavements with surfaces between 50 and 100 mm, use the average daily truck traffic count, for it is probably the best measure of the number of applications of load.

Cognizance should be taken of the fact that under fairly thick surfaces (100 mm or greater) single wheels with high wheel loads appear to cause the greatest degree of fatigue damage per application. Steps should be taken to minimize the number of vehicles of that type on pavements that are similar to the experimental pavement because those vehicles cause more damage than, for example, overloaded axles with dual-wheel configurations.

Fatigue Prediction Procedures

The NIRR fatigue prediction procedure in combination with the University of California method is recommended for use in predicting the fatigue behavior of bituminous layers of all practical thicknesses because it accounts for the complex effect of material properties, structural characteristics, and also traffic and environmental conditions. A more elementary procedure such as the University of Nottingham method can also be used to obtain an approximate prediction of fatigue, a result that may be acceptable in practice. The Shell procedure, however, is not recommended for accounting for the effects of fatigue in surfaces less than 100 mm thick.

Fatigue Initiation and Crack Propagation

It has been found that the NIRR fatigue prediction procedure gives important practical indications of how thin surfaces will behave. If a particular pavement has a short fatigue life, the stiffness of the surface will deteriorate very rapidly in the crack-propagation zone. On the other hand, if the fatigue initiation period is long, the surface stiffness will deteriorate slowly.

The implication is that, if cracks are observed in a pavement surface soon after construction, remedial action should be equally prompt; but, if they appear a long time after construction, immediate action is probably unnecessary. The results on the experimental pavement indicated that equal time periods are required to initiate fatigue and to reduce the surface stiffness to approximately 50 percent of its original value.

ACKNOWLEDGMENTS

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Discussion

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The authors are to be complimented for an excellent paper presenting a wealth of information useful for the practical design of thin bituminous surfaces. Two of their conclusions that the fatigue of thin surfaces should be predicted on the basis of the actual number of axles, irrespective of axle loads, and that fatigue may be minimized by keeping the surface as thin as possible are quite interesting and appear to be reasonable. However, there are situations under which those conclusions may not be valid. The purpose of this discussion is to present numerical data from 2-layer elastic theory and point out the limitations of those 2 conclusions.

It is not clear to the writer what kind of wheel loads, i.e., single, multiple, or a combination of both, the authors used in fatigue prediction. If single wheel loads were employed, did they consider only the principal tensile strain directly beneath the center of the circular loaded area or, more important, the maximum principal tensile strain, which, in thin pavements, generally occurs at some distance from the center? Based on their statement that for 25-mm surfaces the radial strain directly beneath the wheel load decreases with an increase in wheel load, becoming compressive at high loads, it is quite probable that they did not consider the most critical tensile strain in the prediction of fatigue. Although the radial strain directly beneath the wheel load may become compressive in a thin surface under a heavy load with a large contact area, considerable tensile strain may be developed at some other points, which should certainly be used in the prediction of fatigue.

Figure 14, based on the conventional 2-layer elastic theory, shows the relation between the thickness of bituminous surface h_1 and the principal tensile strain e at various distance r from the center of a circular loaded area when both layers are assumed incompressible with a Poisson's ratio of 0.5. The strain, thickness, and radial distance are expressed as dimensionless ratios E_1e/q, h_1/a, and r/a respectively, where E_1 = modulus of elasticity of bituminous surfacing, q = uniformly applied contact pressure, and a = tire contact radius. The modulus ratio $E_1/E_2 = 5$ applies to a thin bituminous surface on an unbound base, and the modulus ratio $E_1/E_2 = 1$ applies to a thin bituminous surface on a stabilized base. For thick surfaces, the maximum tensile strain occurs directly beneath the center of the load, or $r = 0$; for thin

Figure 14. Effect of thickness and radial distance on principal tensile strain.

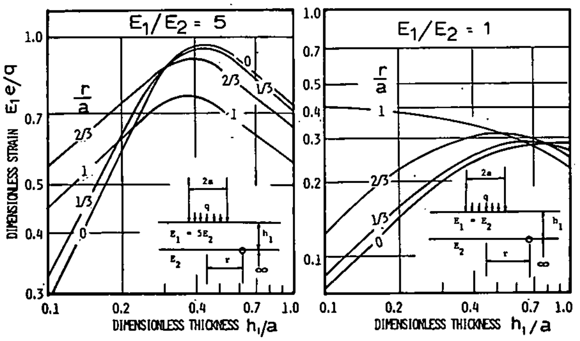

Figure 15. Effect of thickness and modulus ratio on critical tensile strain.

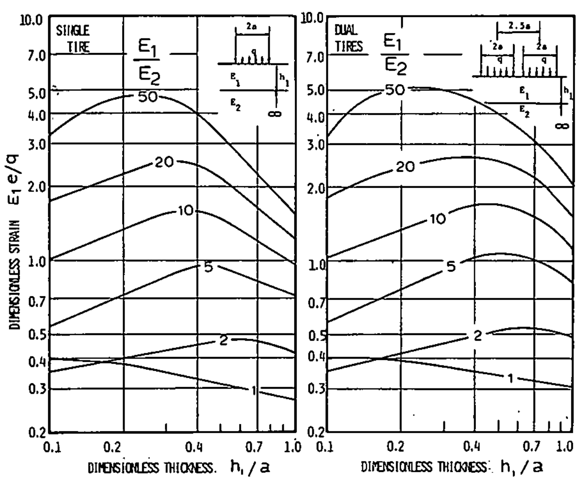

Table 4. Wheel configurations and tire pressures for various wheel-load categories.

Axle Wheel Load (kip)	Mean Wheel Load (kN)	Mean Tire Pressure (bar)	Coefficient of Variation (percent)	Axles Tested	Axles With Single Wheels (percent)	Axles With Dual Wheels (percent)
0 to 4	4.54	1.88	10.5	6	100	0
4 to 8	13.6	5.06	4.2	23	95.7	4.3
8 to 12	22.7	5.53	13.2	177	96.0	4.0
12 to 16	31.7	5.56	14.1	227	25.1	74.9
16 to 20	40.9	5.68	13.5	369	4.0	96.0
20 to 24	49.9	5.86	10.5	90	1.0	99.0
24 to 28	59.0	6.23	8.6	11	0	100

surfaces, it occurs at some distance from the center. In view of the fact that heavier wheel loads are generally associated with greater contact radii and larger contact pressures, the authors' contention that the magnitude of wheel load is immaterial in predicting the fatigue of thin surfaces is valid only when the principal tensile strain at $r = 0$ is used as a fatigue criterion, because the rapid decrease in tensile strain with the increase in contact radius will compensate the increase in tensile strain due to the increase in contact pressure. Because the critical tensile strain does not occur at $r = 0$, the effect of contact radius may not be large enough to compensate for the effect of contact pressure. For a modulus ratio of 1, as shown in Figure 14, the critical tensile strain occurs at $r = a$ and increases with increasing contact radius. In that case, the magnitude of wheel loads will have tremendous effect on the critical tensile strain and, therefore, should not be ignored in the prediction of fatigue.

Figure 15 shows the effect of thickness and modulus ratio on the critical tensile strain under both single and dual tires. Those curves substantiate the authors' finding that the 50-mm surface is more susceptible to fatigue than either the 25-mm or the 100-mm surface. For a given modulus ratio, there is a critical thickness at which the principal tensile strain is maximum. The critical thickness for a typical modulus ratio of 5 is about 0.4a for a single tire and 0.5a for dual tires. Because the contact radius for a conventional 9,000-lb wheel load with a tire pressure of 75 psi is about 150 mm for a single tire and 110 mm for dual tires, it can be easily explained why a thickness of 50 mm is more fatigue susceptible than that of 25 or 100 mm. The figure also shows that below the critical thickness the maximum tensile strain decreases with the decrease in thickness, thus confirming the authors' finding that fatigue may be minimized by keeping the surface as thin as possible. However, that statement is not true when $E_1/E_2 = 1$, or when stabilizing agents are added to the unbound base. In that case, the critical tensile strain increases with the decrease in thickness, and the thinner the pavement is, the more it is susceptible to fatigue.

This discussion indicates that the authors' conclusions are qualitatively correct. However, they do not hold when a thin bituminous surface is constructed on a strong base with a modulus ratio approaching one.

Closure

The authors would like to thank Huang for his interest in our paper, and we will attempt to answer the questions put by him.

1. In the prediction of fatigue life, the kind of wheel loads used were taken from a survey of the types of vehicles that were likely to use the facility. The wheel configurations and tire pressures were measured in 1968 for specific wheel-load categories. The data obtained in those measurements are given in Table 4.
2. The variation in maximum tensile strain with radial distance from the load centerline was recognized as being of primary importance and was taken into account in the computations. Greater details of the tests made on the experimental pavements and the related computations have been presented in a separate publication by the authors (19).
3. With respect to the limitation on our proposals in that they do not hold when a thin bituminous surface is constructed over a strong base with the modulus ratio approaching one, it is considered that in practice this will not constitute a severe limitation because fatigue of the bituminous layer is unlikely to be a critical factor in a pavement of that type. For example, Pell and Brown (37) have indicated that the dynamic modulus of elasticity of cement-bound material such as lean concrete is about 6,900 to 34,500 MN/m^2. Under those conditions and taking $E_1/E_2 = 1$, $h_1/a = 0.1$, and $q = 5.5$ bar for both single and dual wheels, the critical tensile strain ranges from 31×10^{-6} to 6×10^{-6} strain. Tensile strains of that order of magnitude will lead to very

long fatigue lives of the thin bituminous surface irrespective of whether or not the strain increases with load.

Reference

37. Pell, P. S., and Brown, S. F. The Characteristics of Materials for the Design of Flexible Pavement Structures. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972, p. 335.