STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENTS TO PREVENT FATIGUE CRACKING

REFER TO:	ACT	INF.	SIG.
MATERIALS ENGR.		1	[illegible]
OFFICE MANAGER			
QUALITY CONTROL		7	[illegible]
PROJ. RECDS. & SUMM.			
ASSOC. MATLS. ENGR. II		2	[illegible]
PROJ. DEVELOPMENT			
SOILS & FOUND. ENGR.		3	[illegible]
SOILS			
SOIL MECHANICS			
CHIEF GEOLOGIST		4	[illegible]
DRILL CREWS			
ASSOC. MATLS. ENGR. I		5	[illegible]
RESEARCH		8	
TESTING ENGR.		6	[illegible]
CHEMISTRY			
ASPHALT			
STRUCTURES			
ASPHALT MIX			
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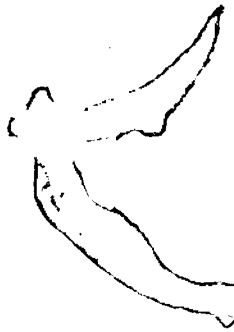

SPECIAL REPORT 140
Highway Research Board · National Research Council · National Academy of Sciences-National Academy of Engineering

Highway Research Board 1973

Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Cracking

Proceedings of a symposium held January 22, 1973, during the annual meeting of the Highway Research Board

Subject Areas

25 Pavement Design
26 Pavement Performance
31 Bituminous Materials and Mixes
62 Foundations (Soils)
63 Mechanics (Earth Mass)

Special Report 140
HIGHWAY RESEARCH BOARD
National Research Council
National Academy of Sciences–National Academy of Engineering
Washington, D.C., 1973

Notice

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Contents

Foreword

The Highway Research Board Task Force on Structural Design of Pavement Systems sponsored a Symposium on Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Failure at the Highway Research Board's fifty-second annual meeting. At the symposium, papers were presented on recent materials and pavement research on the problem of load-associated cracking of asphalt pavements.

Papers by Barksdale and Hicks, Pell, Hudson, and Deacon summarize the results of recent research that can be implemented according to present design practice. Included are (a) examination and evaluation of currently available techniques for evaluating the properties of materials used in asphalt pavement structures and for determining the response of pavement structures to load in terms of stress, strain, and deflection; (b) discussion of current methods for defining fatigue of asphalt mixtures and summary of test results to date; (c) discussion of necessary environmental and traffic inputs to examine fatigue; and (d) summary of available techniques for fatigue-life estimation.

Finn's paper summarizes available information relating load-associated cracking and its areal extent to pavement performance. Terrel describes a series of studies of test tracks and in-service pavements in which the fatigue phenomenon was investigated.

Two design procedures that include the fatigue factor illustrate that these concepts are workable and can be implemented. Havens, Deen, and Southgate describe a procedure applicable for highway pavement design in Kentucky, and Witczak describes a procedure for thick-lift asphalt concrete airfield pavements.

The paper by Majidzadeh and Ramsamooj is an example of promising future developments in this area.

The paper by Freeme and Marais, although not given at the symposium, has been added to this Special Report because it presents another example of the use of the fatigue subsystem in practice.

The paper by Witczak uses fatigue criteria developed by Kingham from analyses of the performance of selected sections of the AASHO test road. For purposes of information, the paper by Kingham in which those analyses are described is included in this Record.

The introductory paper by Monismith shows in more detail how all of the material that was presented at the symposium can be put together for implementation and includes a design example. Whether the detailed procedure is followed as in the example or whether a simplified process is used will depend on the specific project. In effect, the amount of effort devoted to the analysis (design) can be tailored to the project. That point is illustrated in the design examples presented.

McMahon summarizes the symposium, discusses the role of the Federal Highway Administration in pavement design research, and reviews some of the FHWA-sponsored research projects.

Results of research presented at the symposium demonstrate that asphalt pavements can be designed to minimize cracking from repeated traffic loading. The Task Force on Structural Design of Pavement Systems is pleased to see that the Federal Highway Administration is planning to produce in the near future a complete design procedure to investigate the potential for fatigue cracking. Material presented at the symposium should assist in that development of a workable fatigue design system.

The best way to get research into practice, of course, is for people to use it where it seems appropriate even though more "engineering" may be required. The task force believes that those who use the concepts presented at the symposium will increase their capabilities to design with confidence. Engineers who are concerned with pavement design are urged, therefore, to use these techniques on specific projects in conjunction with current procedures.

—C. L. Monismith

Pavement Design: The Fatigue Subsystem

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In the report of the workshop on the structural design of asphalt concrete pavement systems (1), it was indicated that the most frequently occurring mode of distress in asphalt highway pavements in the United States is fatigue cracking associated with traffic loads. That report also indicated that sufficient information is available to permit estimation of that type of distress as a part of the design process, and the recommendation was made that that information be used to check proposed designs. To assist the designer to make such an estimate, this symposium was organized. Its primary objective was to demonstrate that a fatigue subsystem, such as the one shown in Figure 1, of the pavement design and management system is a design tool that can be implemented now. Accordingly, the symposium assembled a series of papers that attempt to bring together the necessary information required to ascertain the potential for fatigue cracking.

Already a number of agencies are using some of the techniques discussed as a part of their design systems. For example, Shell has incorporated the fatigue factor in its design procedure for highway pavements since 1963 and for airfield pavements since 1970. Two working procedures recently developed, one by Havens, Deen, and Southgate in Kentucky for highway pavements and the other by Witczak at The Asphalt Institute for airfield pavements, are included in this Special Report.

It should also be noted that fatigue cracking received considerable attention at the Third International Conference on the Structural Design of Asphalt Pavements; the results presented at that conference (2) reinforce the recommendation made at the HRB workshop (1).

This paper summarizes the steps required to determine whether a particular pavement section will be subject to fatigue from a series of traffic loads (applied during some specified time period). Each box shown in Figure 1 is considered in a stepwise progression, and authors whose papers in this Special Report contain detailed information are indicated so that the basis for a particular step is apparent.

An example is included to illustrate the application of this procedure to determine the potential for fatigue cracking in an existing pavement consisting of an asphalt concrete layer and a base and subbase composed of untreated granular material.

Essentially the process consists of checking a specific section (which may be developed by an existing design procedure such as the AASHO Interim Guides or the California procedure for highway pavements or the Corps of Engineers method for airfield pavements) to ascertain the potential for fatigue distress. If the analysis indicates that fatigue cracking may occur sooner than desired, the design may then be modified. In effect, that process parallels the conventional structural engineering approach in which a structure is selected (designed), its behavior under anticipated service conditions is analyzed, and its adequacy with respect to a specific distress criterion is determined.

In the procedure outlined here, the state of knowledge does not permit an estimate of the time (or amount of traffic loading) to cracking of some specific areal extent. (The paper by Finn discusses the relation between cracking and performance and indicates that at this stage there is no definitive relation between the areal extent of cracking and performance.) Rather, if the estimate is based on laboratory tests (e.g., those discussed by Pell), the time or traffic carried will be that associated with crack initiation; if, on the other hand, the estimate is based on fatigue curves developed from analysis of test roads (Witczak) or in-service pavements (Havens, Deen, and Southgate), then the amount of cracking should be somewhat comparable to that observed in the pavements from which the data were obtained. The approach taken here is a phenomenological one based in part on the principles of continuum mechanics. Although it cannot be implemented at this time, fracture mechanics concepts should assist in the prediction of areal cracking; and thus the topic has been included (Majidzadeh and Ramsamooj) as an example of a promising future development in that area of design.

A USABLE FATIGUE SUBSYSTEM

The information presented in this Special Report provides the necessary basis for the engineer to reasonably ensure that his asphalt pavement structure will carry the traffic estimated to be applied to the pavement within some time period. This section briefly summarizes the steps shown in Figure 1; information presented in the symposium papers is used as a basis.

Traffic

For highway pavements the following traffic information (Fig. 1, box 1) should be determined: axle load distribution; wheel and axle configurations of representative vehicles using facility; contact (or tire) pressures of the various classes of vehicles; distribution of truck traffic throughout day, month, and year; vehicular velocities; and lane distribution of truck traffic for multilane facilities. For airfield pavements, the required information includes gear configurations of representative aircraft using facility, contact (or tire) pressures of the various aircraft, aircraft weights as affected by length of flight and takeoff and landing operations, daily and seasonal variations in aircraft movements, lateral distribution of loads on taxiways (particularly) and runways and longitudinal load distribution on runways, and aircraft velocities.

Methodology for estimating traffic is discussed by Hudson, and Deacon suggests a procedure for discretizing the spectrum of loads to make the traffic factor more manageable in the analysis stage. To simplify the process even further, Havens, Deen, and Southgate use the concept of equivalent axle loads, reducing all traffic to a common parameter—passages of an 18,000-lb axle load. Witczak has made use of the same concept for airfield pavements, defining all aircraft in terms of equivalent passes of a fully loaded DC-8-63F aircraft.

Although lateral distribution of traffic across a lane of a highway pavement is of little import at this time, the lateral distribution of aircraft on both taxiways and runways should be considered to ensure an economical design; Witczak demonstrates how that may be accomplished.

Although the concept of equivalent passes (or equivalent axle loads) is used in the 2 design procedures that have been included, that simplification need not be made. As noted above, Deacon illustrates how the spectrum of loading can be reasonably analyzed.

Environment

Because the response of asphalt-bound materials is dependent on temperature, distributions of temperature within layers containing such materials should be determined. In addition, the influence of environment (Fig. 1, box 1) as it influences the water contents (or effective stresses) of materials constituting pavement sections should be ascertained. In nonfrost areas such determinations will be concerned with potential seasonal changes in water contents (or effective stresses) of the various materials in the pavement structure; in frost areas the effects of freezing and thawing must be considered as well.

Temperature

Pavement temperatures can be computed from weather data (Hudson). That is done by solving the heat conduction equation by numerical techniques, such as finite-difference procedures or finite-element procedure, or by closed-form techniques as presented by Barber. Alternatively, a representative temperature can be estimated by the procedure suggested by Havens, Deen, and Southgate or by Witczak.

Of the computation procedures, the finite-difference method appears to be the most versatile and is recommended for use with layers of different conductivities and for areas where freezing occurs.

All of the available data indicate that temperatures can be reasonably estimated, and there is, therefore, no deterrent to considering that environmental factor in design.

Moisture

One of the most important environmental effects is that of water, particularly because it influences the response of materials in the pavement section to load and because it may cause undesirable volume changes (the latter was not of concern in this symposium).

For design purposes, the influence of water may be considered by measuring the properties of materials at water contents that are assumed representative of those that may develop at some time subsequent to construction, generally a saturated condition (Havens, Deen, and Southgate). In some instances, such procedures result in soil conditions that are not representative of those that develop in the field. Accordingly, it is desirable to have alternative procedures available that provide the designer with estimates of expected in situ moisture conditions and an indication of how those conditions might develop. For example, the procedure might be some measure of the rate of increase (or decrease) in water content of the subgrade soil with time or seasonal variations in the water content of untreated granular materials.

For semi-arid areas or portions of the lower half of the United States where little or no freezing of the subgrade occurs, soil moisture suction considerations should provide a useful (and practical) approach to estimating the equilibrium moisture conditions in fine-grained soils underlying pavement sections with treated layers resting directly on the subgrade (that is briefly discussed by Hudson).

For conditions where freezing and thawing can occur, recent work by Bergan (3) provides a means of defining the subgrade characteristics for use in design.

Construction Requirements and Effects

The influence of construction (Fig. 1, box 4) is not well defined at present and represents an area in which effort should be placed. [Sherman details available information on variations that can occur in paving materials and paving structures (4).] The influence of the degree of compaction on the stiffness characteristics of both treated and untreated materials can be determined (Barksdale and Hicks), and the influence of void content of asphalt-bound materials on their fatigue response can be approximated (Pell). Those effects can be incorporated into the design process as will be seen subsequently.

Design Structural Section

In the design of the structural section (Fig. 1, box 5), one can begin with a pavement cross section selected by an existing method of pavement design. Alternatively, a series of sections can be selected as described by Havens, Deen, and Southgate and by Witczak, and the specific section meeting all of the criteria considered in the design procedure (including fatigue) can be selected.

Materials Considerations

Available materials (Fig. 1, box 2), design asphalt concrete and other treated materials (Fig. 1, box 6), test asphalt concrete (Fig. 1, box 7), and test other pavement materials (Fig. 1, box 8) are interrelated. Related to them in the design process are the following: survey of subgrade soils traversed by the proposed route, selection of the most economical materials to be used in the pavement sections, and design of the asphalt concrete mixture and other treated materials. (By design is meant the selection of mix proportions, e.g., amount of asphalt in the case of asphalt-bound materials.) Emphasis in this symposium was placed only on those aspects of materials characterization necessary to examine the fatigue mode of distress. However, as noted by Pell, the design of the asphalt-concrete mixture to be used is related to the type of pavement in which the materials are used. Accordingly, it must be emphasized that mixture design and structural pavement design must be treated as part of the same process. Figure 1 shows that in stepwise progression.

Characterization procedures for asphalt concrete (Fig. 1, box 7) and the other paving materials (box 8) are discussed by Barksdale and Hicks. For untreated materials, a measure of stiffness (Deacon uses "measure of deformability" in the same connotation) termed the resilient modulus can be determined from a simple repeated load triaxial compression test defined as

$$E_R = \frac{\text{repeated axial stress}}{\text{recoverable axial strain}}$$

Such a technique is suggested by Witczak as one alternative for determination of subgrade stiffness in The Asphalt Institute design procedure.

If equipment necessary to determine such a modulus is not available, estimates from conventional tests can be used as noted by Barksdale and Hicks; i.e. (from Shell), E (in psi) = 1,500 CBR. The Kentucky procedure (Havens, Deen, and Southgate) makes use of that method, and Witczak suggests it as an alternative in The Asphalt Institute procedure. Witczak also suggests the use of approximate relations between E and the measure of stiffness determined from a plate-bearing test if such is available for an airfield pavement design.

For untreated granular materials, the same type of test can be used to determine a stiffness modulus as noted by Barksdale and Hicks. That modulus is dependent on the applied stresses

$$E_R = K \cdot \theta^n$$

where $\theta = \sigma_1 + \sigma_2 + \sigma_3$ (sum of principal stresses).

Alternatively, one can make use of the procedure suggested originally by the Shell investigators wherein the stiffness of the granular layer is proportional to the stiffness of the underlying material; i.e.,

$$(E_{gran}) = F(E_{subgrade})$$

where F = f(thickness of granular layer). In the Kentucky procedure, F is a function of the stiffness of both the subgrade and the asphalt-bound layer varying from 1 to 4 (Fig. 2, Havens, Deen, and Southgate).

Environmental influences must be considered when the response of untreated materials is assessed. Accordingly, proper water contents (or suctions) as well as the effects of freezing and thawing should be reflected in the stiffness measurements.

For asphalt-bound materials, stiffness as defined by the relation

$$S(t, T) = \frac{\sigma}{\epsilon}$$

where

$S(t, T)$ = mixture stiffness at a particular temperature and time of loading and
σ, ϵ = applied stress and strain,

can be used and either measured directly (Barksdale and Hicks), estimated by the Shell procedure, or estimated by a procedure such as that suggested by Witczak.

The stiffness characteristics of other treated materials are also briefly discussed by Barksdale and Hicks. It can thus be seen that sufficient guidelines are available to characterize materials for use in the fatigue subsystem.

Structural Analysis

To permit estimation of the potential for fatigue distress (Fig. 1, box 9) requires an estimation of the stress and deformations resulting from moving wheel loads on realistic representations of pavement structures. As noted in a number of the papers (Barksdale and Hicks; Deacon; Terrel; Havens, Deen, and Southgate; and Witczak), the assumption of the pavement responding as a layered elastic system appears reasonable at this time. Computer solutions are available (CHEV 5L, BISTRO or BISAR, and GCP-1) to facilitate determination of the stresses.

For the asphalt-bound layer, it must be recognized that, because of the temperature dependency of stiffness and the fact that temperatures vary throughout the day and the year, a simplified representation of that environmental influence must be obtained. Deacon discusses how this might be accomplished for the general case, and Witczak and Havens, Deen, and Southgate present specific procedures.

Determine Fatigue Properties

Considerable evidence is presented indicating that strain is a good damage determinant for fatigue (Fig. 1, box 10) in asphalt-bound materials (Pell; Terrel; Havens, Deen, and Southgate; and Witczak) and that an equation of the following form relates the intensity of strain to repetitions to fracture (Pell):

$$N_f = C\left(\frac{1}{\epsilon}\right)^m$$

Such a relation can be determined from laboratory tests (Pell) or from the analysis of the performance of in-service pavements (Havens, Deen, and Southgate, for Kentucky conditions, and Witczak, who used an analysis by Kingham of the AASHO Road Test pavements). Deacon suggests that a criterion based on the performance of in-service pavements may be more appropriate at this stage because such a criterion would necessarily include some amount of areal cracking.

Data are presented indicating that the distribution function for fatigue lives at a particular stress level can be represented as logarithmic normal (Pell). Use of a function of that type thus permits prediction of not only a mean fracture or service life but also the life corresponding to any desired confidence level.

Pell also notes that the linear summation of cycle ratios appears to be a reasonable cumulative damage hypothesis that permits the prediction of service life for a range in loading conditions from the results of simple-loading laboratory tests.

Fatigue Life Estimation

The paper by Deacon contains a step-by-step procedure whereby the fatigue life (Fig. 1, box 11) of a trial pavement section can be ascertained from the information obtained from the previous steps (Fig. 1, boxes 1 through 10). Essentially the procedure makes use of the linear summation of cycle ratios, which in simple form may be stated as

$$\sum_{i=1}^{i} \frac{n_i}{N_i} = D$$

where

n_i = number of applications at strain level i;
N_i = number of applications to cause failure in simple loading at strain level i; and
D = total cumulative damage.

In that relation, failure occurs when D equals or exceeds 1.0. Thus, the design procedure becomes one of checking the particular section to ensure that D is equal to or less than unity for the anticipated design conditions. [If the value of D is considerably less than one, the section may be underdesigned; when D is greater than one, a redesign (re-analysis) may be in order.] As Deacon notes, various temperatures as well as loading conditions can be considered and thus make the procedure adaptable to any environmental condition.

Terrel presents a number of examples illustrating the applicability of such an approach to analyze fatigue distress occurring in either trial or in-service pavements. The comparisons that he presents lend support to the use of this procedure for design purposes now.

In the design procedure described by Havens, Deen, and Southgate, the fatigue life and thus the design section is associated with the number of repetitions of 18,000-lb single axle load; some equivalency between the other loads and the 18,000-lb value is established. In the procedure described by Witczak, traffic is converted to equivalent passes of a DC-8-63F aircraft for design purposes. In this procedure, use is made of the linear summation of cycle ratios described above.

It must be emphasized that the estimate made by this technique is associated with no specific amount of cracking. If the analysis is based on laboratory fatigue tests, the traffic will be that associated with crack initiation and will provide a slightly conservative estimate. Comparisons of a number of different design procedures are shown in Figure 2 (5) to illustrate this point.

EXAMPLE

The example presented here illustrates the use of the procedure to check a pavement designed by the California procedure to ascertain whether it will be adequate to resist fatigue cracking for the 10-year period for which it was designed. The example is presented primarily to illustrate the concepts discussed at the symposium and has purposely been made brief. Details of the procedure may be found elsewhere (6).

The structural pavement section of this project east of Sacramento, California, on US-50 consists of 0.33 ft of asphalt concrete, 0.25 ft of asphalt-treated base, and 1.50 ft of untreated base and subbase on a subgrade of compacted dredger tailings.

Traffic

Data from the Perkins recording station, located near the test section, was obtained from the California Division of Highways. Average daily traffic for 1970 was reported to be as follows:

Vehicles	ADT	Percent of Trucks
All	25,000	
Trucks	1,400	100

Vehicles	ADT	Percent of Trucks
2 axle	800	57
3 axle	100	7
4 axle	100	7
5 or more axles	400	29

The Folsom project was placed in service either in late 1965 or early 1966. Because traffic data are not available for the earlier period, the 1970 data were reduced at a rate of 3 percent per year to obtain the 1966 ADT. Resulting estimates for trucks are as follows:

Axles	Number	Percent
2	708	57
3	87	7
4	87	7
5	223	18
6	137	11
Total	1,242	100

Truck traffic distribution throughout the day in hourly increments was estimated by the use of the average annual statewide data given in Table 1. Axle load groups (12 in this analysis) were related to operations in each classification by means of the monthly wheel load factors given in Table 2. Those factors permit the determination of wheel loads of a specific magnitude in each axle category.

Additional assumptions were as follows:

1. Truck operations are equally divided directionally;
2. Eighty-five percent of the operations in each direction occur in the lane adjacent to the shoulder (design lane); and
3. Truck operating speeds correspond to a time of loading of 0.02 sec in the asphalt concrete.

Analytically the traffic variables were related as follows:

$$AHT_{ij} = ADTT \cdot \frac{A_j}{100} \cdot \frac{HT_i}{100}$$

where

$ADTT$ = average daily truck traffic, 1 direction in design lane;
A_j = percentage of truck traffic of class j (2 axle, 3 axle, and so on);
HT_i = percentage of truck traffic in the hourly interval i (Table 1); and
AHT_{ij} = number of operations of class j in hour i (daily).

$$AXLD_{ik} = \sum_{j=1}^{S} (AHT_{ij} \cdot WLF_{kj})$$

where

WLF_{kj} = monthly wheel load factors to relate axle class j to axle load group k (Table 2); and
$AXLD_{ik}$ = matrix of the number of axle loads of group k in each hour i (on a monthly basis.

The $AXLD_{ik}$ values were expanded to an annual basis by incorporating climatic information as described in a subsequent section.

Environment

The climatic data used for the Folsom area (Table 3) were obtained from records

Figure 1. Diagram of a fatigue subsystem.

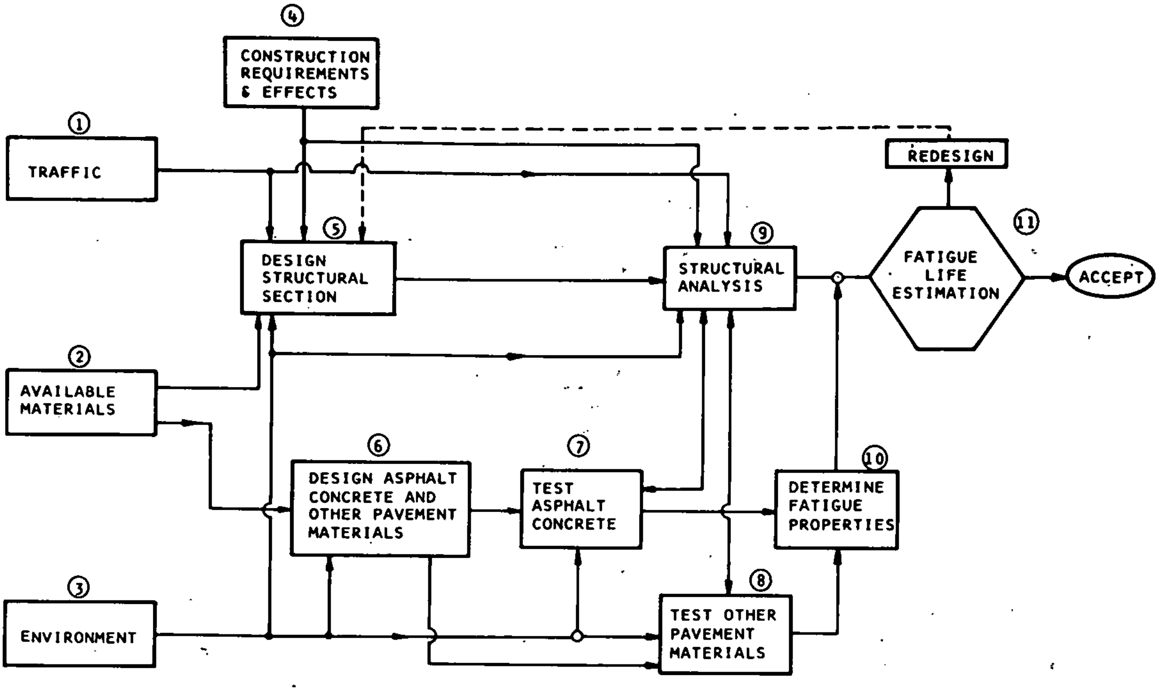


Figure 2. Comparison of thickness of asphalt concrete determined by existing procedures.

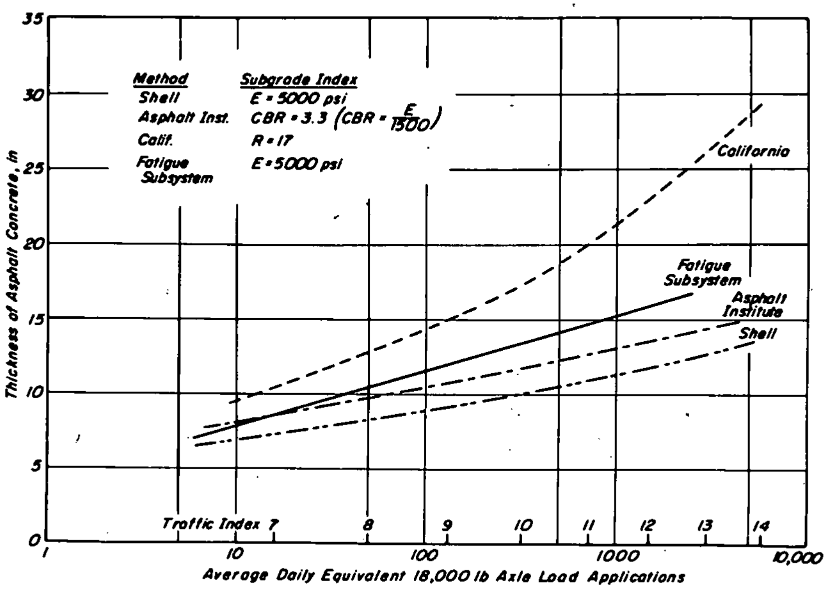


Table 1. Annual average daily statewide truck traffic in 1967.

Hour	Percent of Traffic	Hour	Percent of Traffic
12 to 1 a.m.	2.8	12 to 1 p.m.	5.2
1 to 2	2.7	1 to 2	5.6
2 to 3	2.9	2 to 3	5.7
3 to 4	3.1	3 to 4	5.7
4 to 5	3.5	4 to 5	5.4
5 to 6	4.1	5 to 6	4.5
6 to 7	4.2	6 to 7	3.8
7 to 8	4.3	7 to 8	3.5
8 to 9	4.8	8 to 9	3.3
9 to 10	5.0	9 to 10	3.2
10 to 11	5.2	10 to 11	3.3
11 to 12	5.2	11 to 12	3.0

Table 2. Monthly wheel load factors based on W-4 loadometer studies from 1966 to 1968.

Axle Load (kips)	2-Axle	3-Axle	4-Axle	5-Axle	6 or More Axles
Under 3	4.126	0.161	1.034	0.511	1.450
3 to 7	17.786	16.425	20.904	19.658	21.740
7 to 8	2.083	6.342	7.498	6.785	8.172
8 to 12	3.816	12.814	18.334	16.777	32.766
12 to 16	1.503	6.955	8.203	16.669	19.998
16 to 18	0.537	1.927	3.322	12.947	4.459
18 to 20	0.142	0.340	0.627	1.934	0.585
20 to 22	0.004	0.033	0.101	0.058	0.724
22 to 24	0.003	0.004	–	0.027	0.227
24 to 26	–	–	–	0.006	0.204
26 to 30	–	–	0.009	0.011	0.399
30 to 35	–	–	–	–	–

made at the Sacramento airport, the nearest location to the test site for which records were available for long periods of time.

Materials Characterization

Four sampling locations were selected, and slabs of asphalt concrete were taken for laboratory measurements of stiffness.

Granular base and subbase materials were obtained at the same locations to determine in-place densities, water contents, and gradations. Additional samples for preparation of laboratory test specimens were obtained from the plant supplying the materials to the job site. Subgrade samples were also obtained from the 4 test locations for in situ densities, water contents, and gradations.

Subgrade Soil

Subgrade soils were described as a combination of weathered slate, lava conglomerate, and silty clay at 2 of the sampling locations and as dredger tailings, red silty clay, and cemented cobbles at the other 2 locations. The descriptions were supplied by staff of the Materials and Research Department, July 1967. Atterberg limits for the materials are as follows:

Location	Liquid Limit	Plastic Limit	Plastic Index	Unified Classification
1	33	20	13	CL
3	25	19	6	CL
4	30	29	1	ML

Samples for laboratory resilience testing were prepared by separating the soil into 3 fractions: passing No. 4 sieve (approximately 70 percent), $^3/_4$ in. by No. 4 (approximately 10 percent), and retained on $^3/_4$ in. sieve (approximately 20 percent). The weight of material passing the No. 4 sieve equaled that of the field samples, while material greater than $^3/_4$ in. was wasted and replaced with an equivalent weight of $^3/_4$ in. by No. 4 material. The modified material was then compacted by kneading compaction into 4-in. diameter by 8-in. high specimens, and a series was prepared to cover the range of dry densities and water contents observed in the field sampling program.

Repeated load tests were conducted using a constant cell pressure of 3 psi and a repeated deviator stress ranging from 1 to 5 psi applied at a frequency of 20 repetitions per minute and a duration of 0.1 sec. Resilient moduli were ascertained from recoverable deformations measured over the center 4 in. of the specimens with dual LVDT's clamped to the membrane surrounding each specimen after 1,000 stress repetitions.

From the tests (Table 4) isolines of resilient moduli could be developed (Fig. 3) for a repeated deviator stress of 2 psi. The development of such relations for a range in deviator stresses makes it possible to define a modulus versus deviator stress relation for any specified water content and dry density.

Subbase

The subbase material was a well-rounded gravel conforming to California specifications for a class 1 aggregate subbase.

Prior to specimen preparation, all material was separated into individual fractions by dry sieving, and material retained on the $^3/_4$-in. sieve was wasted. As with the subgrade soil, the percentage by weight passing the No. 4 sieve was maintained for the laboratory specimens the same as for the field samples. In addition, the plus $^3/_4$-in. material was replaced by an equivalent weight of $^3/_4$ in. by No. 4 material.

Specimens for the repeated load triaxial compression tests were prepared by vibratory compaction at water contents representative of those existing in the roadway at the time of sampling and to densities corrected from field values to account for the modified grading.

Table 3. Climatic data.

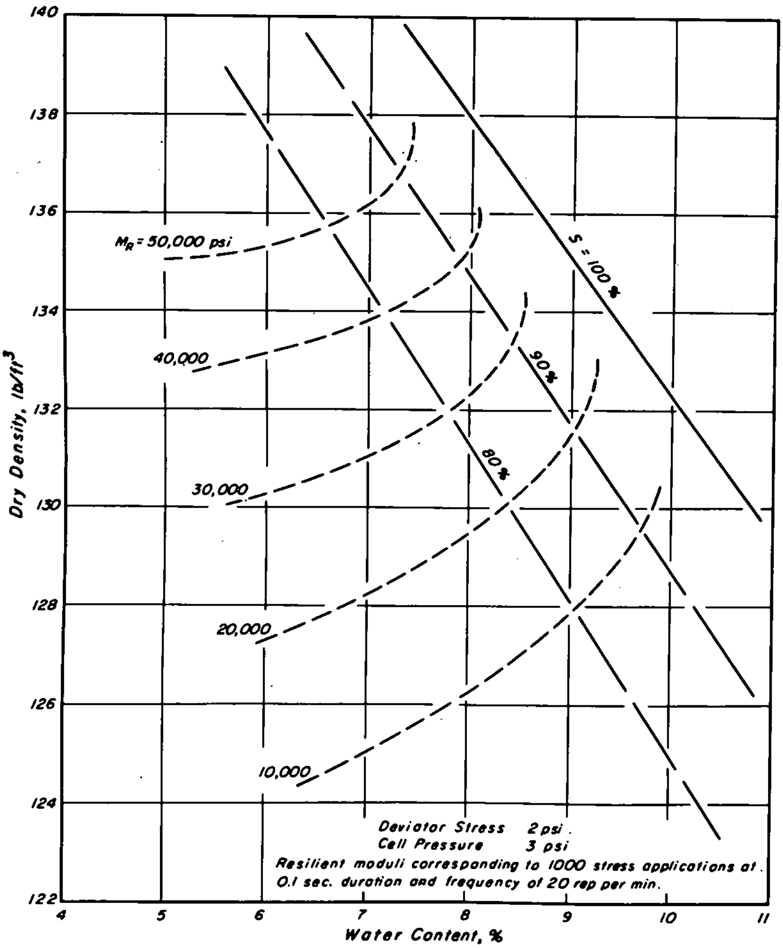
Month	Avg Air Temperature	Daily Air Temperature Range	Avg Wind Velocity	Insolation	Sky Cover
Jan.	45.2	16.0	8.7	182.0	7.1
Feb.	49.2	18.8	8.7	287.0	5.9
Mar.	53.4	22.8	9.7	426.0	5.5
Apr.	58.4	26.1	9.3	547.0	4.6
May	69.0	28.5	9.8	642.0	3.9
June	70.5	32.1	10.5	701.0	2.0
July	75.4	36.0	9.7	685.0	0.9
Aug.	74.1	35.6	9.7	621.0	1.3
Sept.	71.6	33.2	8.3	506.0	1.7
Oct.	63.5	28.2	7.4	374.0	3.2
Nov.	52.9	22.6	6.8	248.0	5.4
Dec.	46.4	16.5	7.4	157.0	6.9

Table 4. Repeated load test results for subgrade soil.

Site	Test	Dry Density (lb/ft^3)	Water Content (percent)	Resilient Modulus (psi × 10^{-3}) 1-psi σ_d	2-psi σ_d	3-psi σ_d	Poisson Ratio at σ_d = 3 psi
1	1	133.0	9.6	2.5	1.9	2.2	–
	2	132.2	9.7	7.8	5.2	4.2	0.43
	3	128.2	10.1	1.5	1.7	2.1	0.45
	4	127.4	7.3	2.0	2.6	2.2	–
2	1	134.1	9.8	–	14.0	14.0	–
3	1	137.4	8.6	16.0	7.6	9.0	0.58
	2	122.8	9.1	15.2	14.3	13.9	0.49
	4	139.0	5.8	63.8	54.0	46.8	0.33
	5	136.2	6.3	95.0	56.0	47.6	–
	6	138.4	7.6	63.8	56.1	55.4	0.42
4	1	130.7	9.3	15.9	19.7	12.4	–
	2	130.5	9.8	5.4	6.1	6.4	0.52
	3	130.6	9.4	8.3	6.4	7.2	0.50

Note: Confining pressure = 3 psi except for test site 2 material where confining pressure = 2 psi.

Figure 3. Water content-dry density-resilient modulus relation for subgrade soil.



The compacted specimens were subjected to a range of repeatedly applied deviator stresses and a range in static cell pressures (σ_3). Both axial and circumferential deformations were measured with LVDT's clamped to the central portion of the specimen. Resilient modulus and resilient Poisson's ratio were determined after 200 axial stress repetitions at a particular state of stress for a particular specimen. The effects of stress history were minimized by first subjecting each specimen to 200 stress repetitions at an intermediate stress ratio, then at a range of stress ratios from low to high, and then at the ratios in reverse.

Linear regression analyses of the logarithm of modulus, M_R versus the sum of principal (total) stresses θ, were obtained for each of the 9 test series and for all of the data together. Although the results for the individual series were generally good (correlation coefficients greater than 0.85), the results for all data together were somewhat poorer and reflected the influence of variations in density and water content.

For the pavement analysis, the equation based on all data

$$M_R = 7730\ (\theta)^{0.46}$$

has been used (correlation coefficient of 0.68).

The dependence of Poisson's ratio on stress was also ascertained, and the following relation has been used in the pavement analysis:

$$\nu = 0.13 + 0.05\ (\sigma_1/\sigma_3)$$

Base

The base material is a crushed gravel conforming to California specification for a class 2 aggregate base.

Resilient moduli were determined in the same manner as for the subbase material, and a linear regression using all the test data resulted in the following relationship:

$$M_R = 3470(\theta)^{0.65}$$

The correlation coefficient was 0.96, indicating less scatter than for subbase data.

For Poisson's ratio, the following relation was established.

$$\nu = 0.16 + 0.08\ (\sigma_1/\sigma_3)$$

Asphalt Concrete Surface

Stiffness characteristics of the asphalt concrete from the pavement specimens were measured in the laboratory at only 1 temperature, 68 F, and 1 time of loading, 0.1 sec. To define the stiffness over the range of temperatures encountered in the field and for a range in loading times, use was made of the Shell procedure and the following recovered asphalt properties for both base and surface courses (control section):

Location	Course	Penetration at 77 F (dmm)	Ring and Ball Softening Point (deg F)
1	Surface	38	132
1	Base	31	135
2	Surface	26	141
2	Base	28	138

The resulting relations between mixture stiffness and temperature for times of loading of 0.02 sec (fast traffic) and 0.1 sec (slow traffic) are shown in Figure 4.

Poisson's ratio was assumed to vary from 0.30 to 0.35 for the temperature range encountered.

In addition, for estimates of the stiffness in situ as a function of temperature using Barber's solution, the asphalt concrete was assumed to have the following thermal characteristics: thermal conductivity = 0.70 Btu/ft^2/hour/deg F/ft; specific heat = 0.22 Btu/lb · deg F; and surface coefficient = 0.85.

Fatigue Characteristics of Asphalt Concrete

Fatigue data for the mixtures used in the project were obtained by testing specimens recovered from the road and by testing laboratory prepared specimens. Those data provide a basis for selecting the appropriate data to be used in the performance estimate.

For the Folsom pavement, which had a 7-in. asphalt concrete layer, stiffnesses were estimated to range from about 100,000 to 2,000,000 psi. For those conditions, the controlled-stress mode of loading provides a conservative estimate of fatigue life. It is probable that an intermediate mode of loading (as defined by the mode factor, 6) would be appropriate. To determine that condition (or conditions), mode factors were computed for a range in asphalt concrete stiffnesses and loads as shown in Figure 5. Computed values ranged from about -0.7 to +0.4 depending on the wheel load and mix stiffness.

Strain versus fatigue life relations for intermediate modes of loading will lie between the limits defined by the controlled-stress and controlled-strain modes of loading. For that analysis, it was assumed that an intermediate mode could be defined by a direct interpolation between the limiting relationships; e.g., the strain-fatigue life relation corresponding to a mode factor of zero would lie midway between the 2 curves.

Some measure of the difference was obtained in controlled-strain tests that were performed at 68 F; results are shown in Figure 6. Comparison of the controlled-stress and controlled-strain data (68 F) are shown in Figure 7. Lines representing the intermediate modes are also shown. The data are based on specimens whose stiffnesses ranged from 160,000 to 170,000 psi. To cover the range in stiffnesses anticipated required some adjustment in the data, and a series of relations like that shown in Figure 7 are required to cover the range of stiffness expected in service.

For example, for a stiffness of 150,000 psi (the lowest value used in the analysis), the controlled-stress relation was steepened slightly and the controlled-strain relation was flattened. Similar adjustments were also made for other stiffnesses. At stiffnesses greater than 700,000 psi, the controlled-stress and controlled-strain lines were assumed to coincide (that decision being based on the experience of other investigators, e.g., Pell).

Structural Analysis

Figure 8 shows the pavement section as well as the material characteristics and other variables considered in the analysis. For convenience the 7-in. asphalt concrete section was considered as 1 layer. The stiffness of the asphalt concrete and the resilient properties of the subgrade were considered to be the primary variables affecting the deflection of the pavement under load. The CHEV 5L (w/iteration) program was used. Because the program only considers a variable modulus in the granular layers, manual iteration was also required to account for the variability in Poisson's ratio.

Deflections computed by this process are shown in Figure 9 as a function of asphalt concrete stiffness. Included are the deflections measured in January 1968. For the temperature conditions at the time of measurement (air temperature = 48 F, and surface temperature = 53 F), the asphalt concrete stiffness was estimated to be about 1,000,000 psi (time of loading of 0.1 sec).

Comparison of the measured deflections with computed values indicates that the dry, high-density subgrade condition is most representative of the in situ condition. Hence, those subgrade properties have been used in the estimate of fatigue life discussed in the next section.

Figure 4. Computed relations between mixture stiffness and temperature.

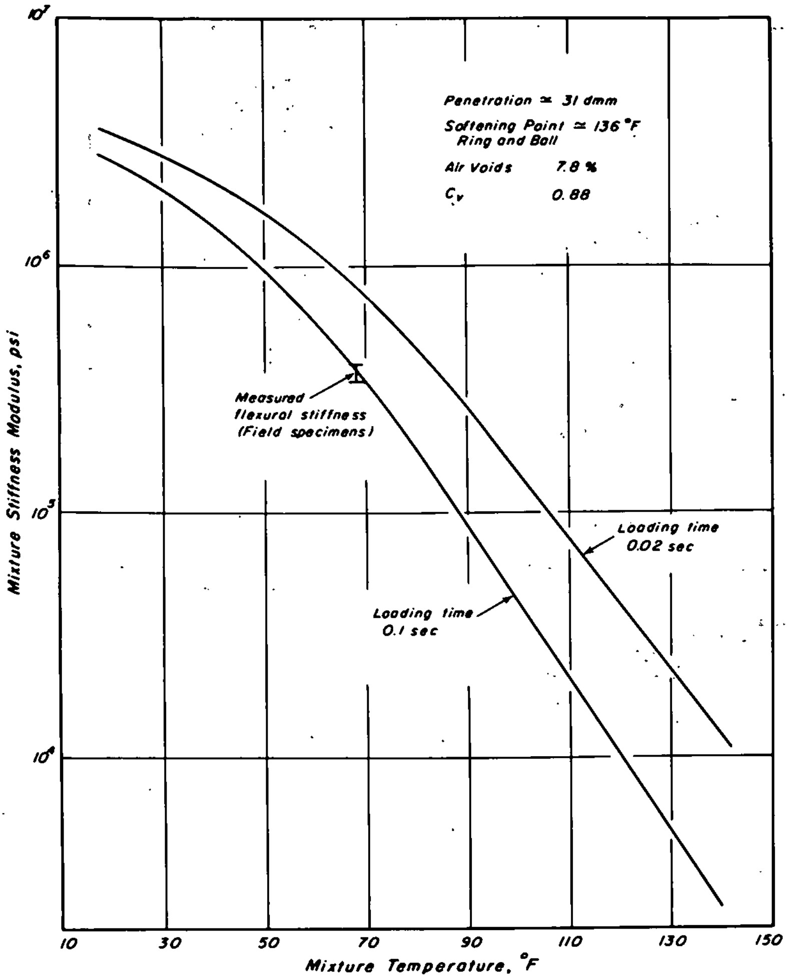


Figure 5. Mode factors as function of asphalt concrete stiffness and axle load.

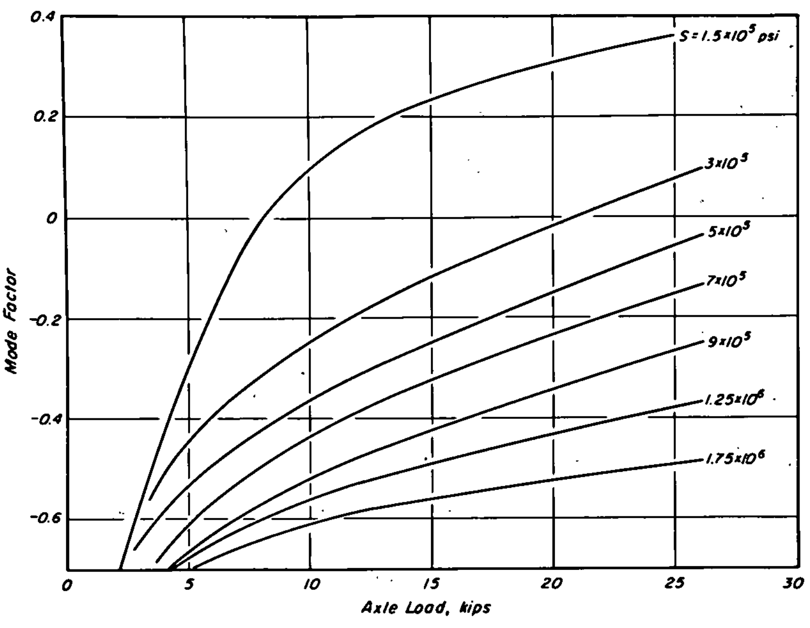


Figure 6. Initial bending strain versus service life in controlled-strain (deflection) tests on laboratory-prepared specimens.

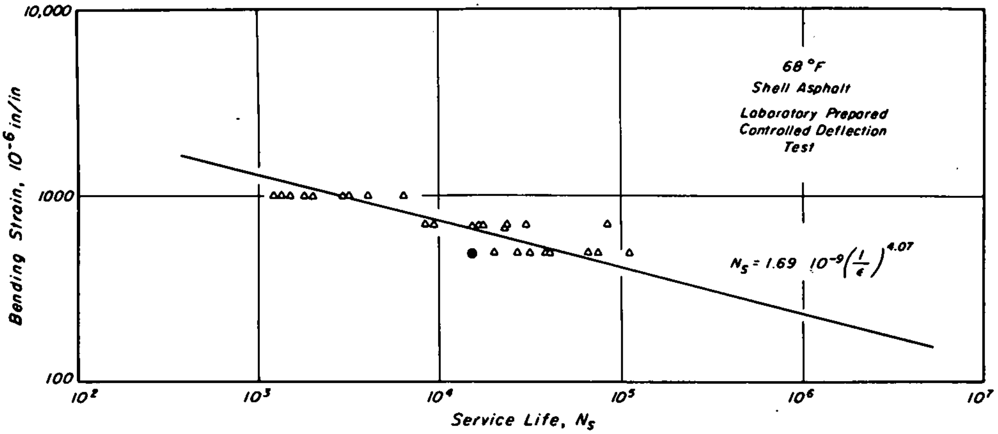


Figure 7. Fatigue curves for series of modes of loading.

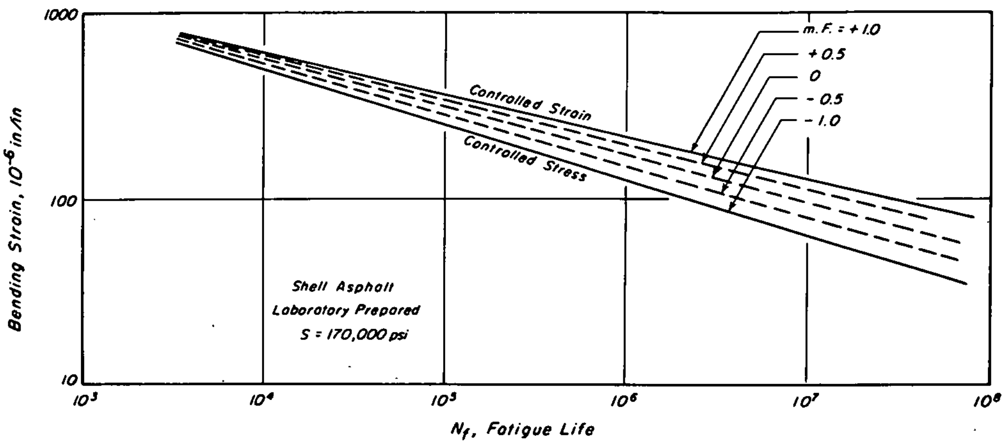
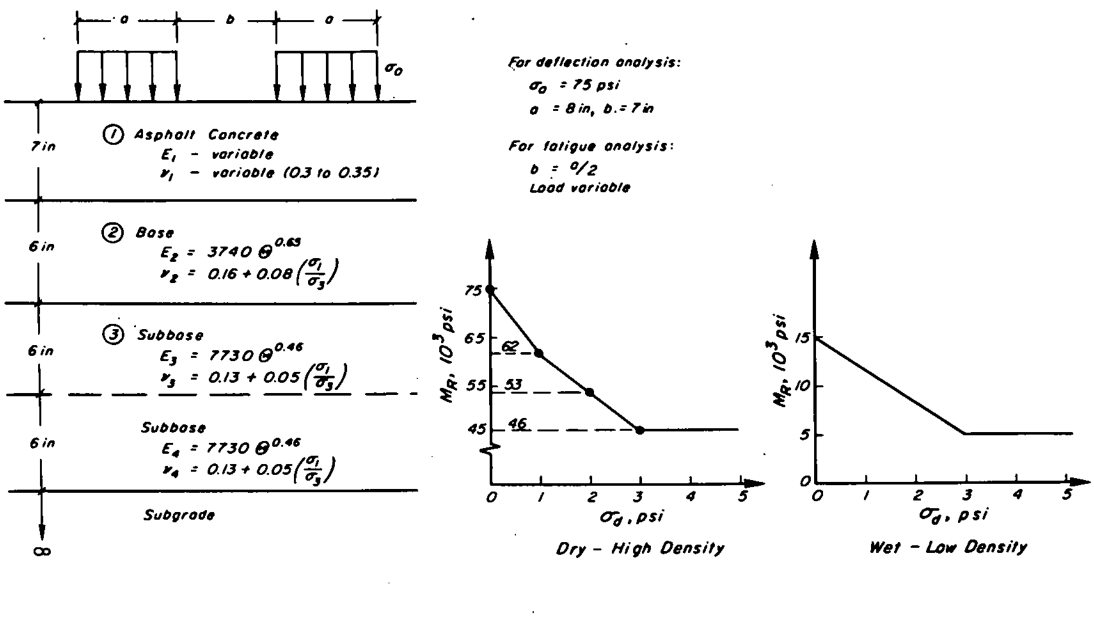


Figure 8. Pavement section used in analysis.



During this analysis, it was observed that the vertical stresses at the subgrade surface were of sufficient magnitude (at least 3 psi) so that the modulus could be considered constant. That simplifying assumption has been used in all subsequent calculations.

To consider the effects of temperature on stiffness, we developed a simplified procedure in which the computer is used. The procedure permits determination of the temperature T_i and stiffness S_i (Fig. 10) at the beginning and end of each hour of the day (25 values per day) at each specified depth d_i.

If w_i is defined as

$$w_i = \left(\frac{d_i - d_{i-1}}{2}\right) + \left(\frac{d_{i+1} - d_i}{2}\right)$$

(w_i and w_n must be adjusted for the boundaries), then w_i is the increment of total depth that each T_i and S_i represent. Means of temperature and stiffness can then be determined.

$$T_{mean} = \left(\sum_{i=1}^{n} T_i \cdot w_i\right) \Big/ D$$

$$S_{mean} = \left(\sum_{i=1}^{n} S_i \cdot w_i\right) \Big/ D$$

Twenty-five such values are computed for each day.

The values for the beginning and end of each hour are averaged, and a representative value of the stiffness of the asphalt concrete during the hourly increment is obtained. Because the rate of change of stiffness in any mass of asphalt concrete is not large, this averaging was considered to be a reasonable estimate of the mean stiffness during the 1-hour interval.

Use of the mean stiffness also permits the full depth of asphalt concrete to be represented by a single modulus. Although there are some advantages to that simplification (as compared to the characterization of the asphalt concrete as 2 or more layers), use of the single value has some limitations. For example, a system consisting of soft-stiff layers versus one of stiff-soft layers, both with the same mean, does not exhibit the same stresses and deformations. However, for the approximately sinusoidal distribution of temperature with time used in the analysis, where the heating period is nearly balanced with the cooling period, differences tend to be compensated.

The computations result in $24 \times 12 = 288$ values of stiffness modulus to represent the daily and seasonal variation of stiffness modulus. Those values were then grouped, and the frequency matrix given in Table 5 was prepared. The frequencies are the number of months that a given interval of 1 hour had a mean stiffness in the group shown.

If these frequencies are designated $f_{i\ell}$, then traffic and stiffness can be related by

$$ADL_{\ell k} = \sum_{i=1}^{24} (f_{i\ell} \times AXLD_{ik})$$

where $ADL_{\ell k}$ = annual number of applications of axle load group k to the pavement when the stiffness occurs in stiffness group ℓ.

Stresses and deformations at the base of the asphalt concrete were determined by the multilayer linear elastic solution for a range of axle loads and asphalt concrete stiffnesses.

Results of those computations are shown in Figure 11 in the form of strain versus axle load with isolines of constant stiffness. Thus, the strain corresponding to each of the 12 axle load groups and each of the 7 stiffness groups can be obtained.

Figure 9. Computed and observed pavement deflections.

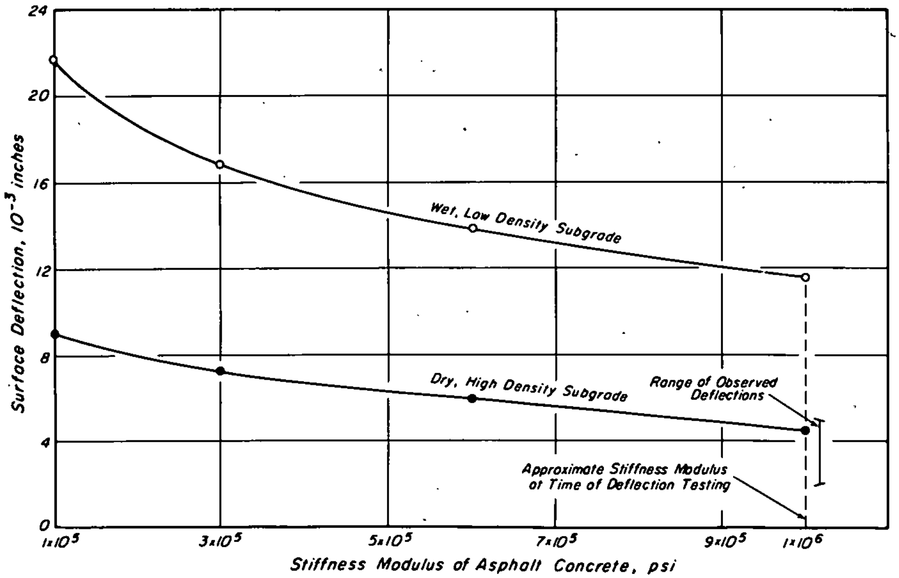


Figure 10. Notation for stiffness and temperature determinations.

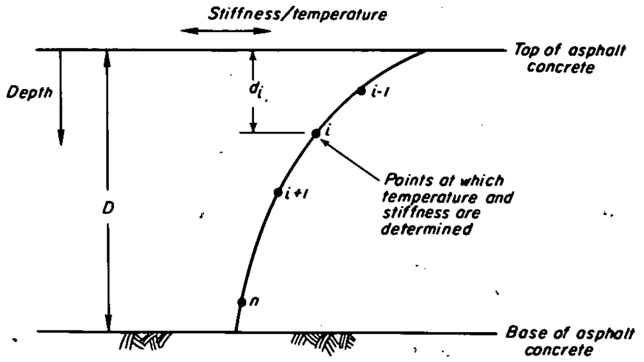


Table 5. Frequency of occurrence of pavement stiffness.

	Stiffness Group (psi × 10^6)						
	1.5 to 2.0	1.0 to 1.5	0.8 to 1.0	0.6 to 0.8	0.4 to 0.6	0.2 to 0.4	0.1 to 0.2
	Midpoint Stiffness (psi × 10^6)						
Time	1.75	1.25	0.90	0.70	0.50	0.30	0.15
12 to 1 a.m.	3	3	1	1	3	1	-
1 to 2	3	3	2	2	2	-	-
2 to 3	3	3	2	2	2	-	-
3 to 4	4	2	2	2	2	-	-
4 to 5	4	2	2	2	2	-	-
5 to 6	4	2	2	2	2	-	-
6 to 7	4	2	2	2	2	-	-
7 to 8	3	3	2	2	2	-	-
8 to 9	3	3	2	2	2	-	-
9 to 10	3	3	2	1	2	1	-
10 to 11	3	2	2	1	3	1	-
11 to 12	3	2	1	2	2	2	-
12 to 1 p.m.	2	3	1	2	1	3	-
1 to 2	2	3	-	2	1	3	1
2 to 3	2	3	-	2	1	3	1
3 to 4	2	3	-	1	2	2	2
4 to 5	2	2	1	1	2	2	2
5 to 6	2	3	-	1	2	1	3
6 to 7	2	3	-	1	2	1	3
7 to 8	2	3	-	2	1	2	2
8 to 9	2	3	-	2	1	2	2
9 to 10	2	3	1	2	-	4	-
10 to 11	2	3	1	2	2	2	-
11 to 12	3	1	2	1	2	2	-

Figure 11. Bending strain on underside of asphalt layer and axle load for stiffness range.

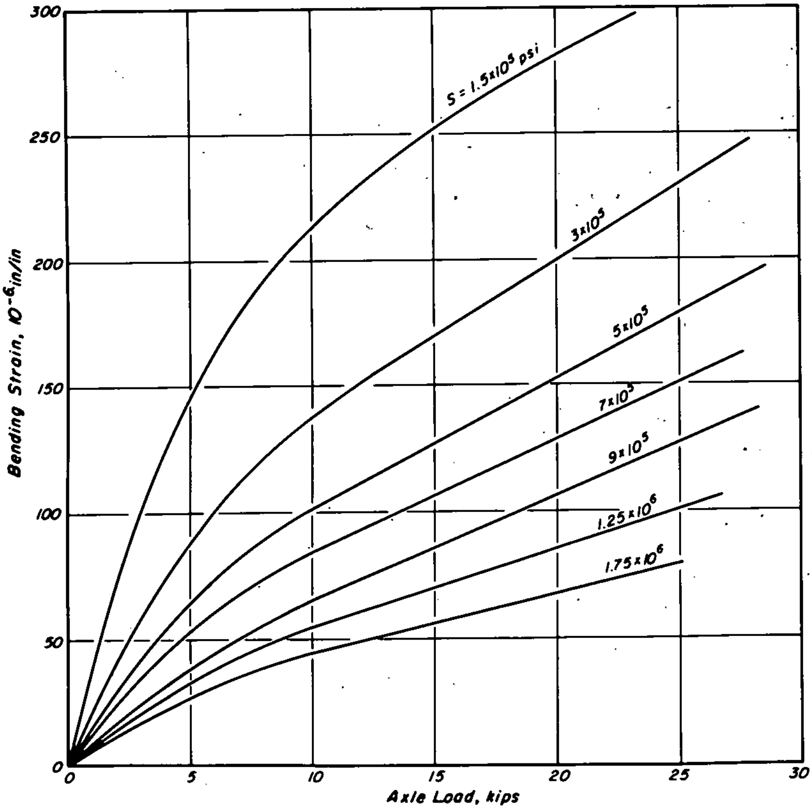


Table 6. Fatigue life matrix.

Axle Load	1.75[a]			1.25[a]			0.90[a]			0.70[a]			0.50[a]			0.30[a]			0.15[a]		
	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]
1.5-1	9	-1	10^{99}	11	-1	10^{99}	13	-1	10^{99}	18	-1	10^{99}	23	-1	10^{99}	30	-1	10^{99}	54	-1	10^{99}
5-2	26	-0.8	10^{99}	33	-0.7	10^{99}	39	-0.65	10^{99}	53	-0.6	10^{99}	64	-0.5	10^{99}	87	-0.4	7.0	146	-0.3	1.6
7.5-3	37	-0.65	10^{99}	46	-0.6	10^{99}	54	-0.6	10^{99}	71	-0.5	6.5	86	-0.45	4.5	116	-0.3	2.4	165	0.0	0.95
10-4	45	-0.6	10^{99}	55	-0.55	10^{99}	65	-0.50	10^{99}	84	-0.45	3.6	102	-0.35	2.7	138	-0.25	1.4	212	0.1	0.72
14-5	54	-0.55	10^{99}	66	-0.50	10^{99}	81	-0.95	2.75	101	-0.35	1.8	122	-0.3	1.45	163	-0.15	0.95	246	0.2	0.38
17-6	61	-0.55	10^{99}	76	-0.50	2.8	93	-0.4	1.7	115	-0.3	1.15	136	-0.2	1.0	180	-0.1	6.0	264	0.3	0.33
19	65	-0.55	10^{99}	82	-0.45	1.95	102	-0.35	1.2	124	-0.25	0.84	148	-0.2	0.71	194	-0.05	0.52	275	0.3	0.29
21	70	-0.5	3.3	88	-0.45	1.5	110	-0.35	0.76	132	-0.2	0.68	158	-0.15	0.58	206	0.0	0.44	285	0.3	0.24
23	75	-0.5	2.5	94	-0.4	1.1	118	-0.3	0.62	141	-0.2	0.54	168	-0.1	0.49	218	0.05	0.33	295	0.35	0.22
25	75	-0.5	2.1	100	-0.4	0.88	126	-0.3	0.50	150	-0.15	0.43	178	-0.05	0.40	232	0.05	0.23	304	0.35	0.20
28	86	-0.5	1.4	108	-0.35	0.64	138	-0.2	0.36	163	-0.1	0.32	194	-0.0	0.28	250	0.1	0.27	318	0.4	0.22

Note: Matrix based on Shell laboratory data where $N_f = \infty$ at $\epsilon < 70 \times 10^{-6}$ in./in.

[a] Stiffness, psi x 10^6.

[b] Except for the values 10^{99}, all other values are x 10^6.

Fatigue Life Determination

Data necessary to complete the fatigue analysis were set up in the form given in Table 6. The strain corresponding to each stiffness modulus and axle load group was first obtained from data shown in Figure 11. A mode factor was then obtained from data shown in Figure 5; the number of repetitions to failure for the specific strain level and mode factor could then be obtained from a series of charts of the form shown in Figure 7. These repetitions were designated $ENNF_{\ell k}$, where ℓ is the stiffness group and k is the axle load group. Then,

$$\sum_{\ell}\sum_{k} (ADL_{\ell k}/ENNF_{\ell k}) = \text{annual damage}$$

Traffic values were then incremented by an expansion factor (3 percent), and the process was repeated until the total damage was equal to one.

Fatigue life predictions were made by the use of 2 sets of strain-repetitions to failure relations.

1. Fatigue life was based on tests of laboratory results. Individual estimated lines were linearly extrapolated to the full range of strain computed. These data resulted in a predicted fatigue life of 9.5 years.
2. The same data were used as in the preceding relation except that the assumption was made that strains less than 70×10^{-6} in./in. caused no fatigue damage (6). The purpose of that calculation was to estimate the cumulative effect that the large number of small loads have on the fatigue life estimate. Those data resulted in a predicted fatigue life of 12 years.

Discussion of Example

In this example, the California Division of Highways had designed the pavement for a 10-year life. Based on the measured traffic data for 1970 and reasonable extrapolations to both the initial and later stages of its service life, the pavement can be expected to perform reasonably from a fatigue standpoint. That is, based on the mean fatigue data, a service life in the range 9 to 12 years appears feasible.

This example demonstrates the feasibility of design to preclude fatigue and illustrates many of the concepts described in detail in the subsequent papers in this report.

SUMMARY

The technology necessary to design airfield and highway pavements to minimize fatigue distress was presented at the symposium. The purpose of this introductory discussion is to develop a framework for the papers and to demonstrate that the results of research efforts during the past 10 years can be put into practice to make reasoned engineering decisions regarding the behavior of pavements relative to load-associated cracking.

The information discussed here holds the potential (as compared with and when used with conventional procedures) to assist the engineer to better utilize available materials; improve the reliability of the design estimate; consider loadings larger than those now applied, if required; better define the role of construction; and analyze existing pavements.

REFERENCES

1. Structural Design of Asphalt Concrete Pavement Systems. Highway Research Board Spec. Rept. 126, 1971.
2. Proceedings. Third International Conference on the Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbor, 1972.

3. Bergan, A. T. Some Considerations in the Design of Asphalt Concrete Pavements for Cold Regions. Univ. of California, Berkeley, PhD dissertation, Sept. 1972.
4. Sherman, G. B. In Situ Materials Variability. Highway Research Board Spec. Rept. 126, 1971, pp. 180-188.
5. Monismith, C. L., and McLean, D. B. Symposium in Technology of Thick Lift Construction: Structural Design Considerations. Proc., AAPT, 1972.
6. Monismith, C. L., Epps, J. A., Kasianchuk, D. A., and McLean, D. B. Asphalt Mixture Behavior in Repeated Flexure. Univ. of California, Berkeley, Rept. TE 70-5, 1971.

This paper examines and evaluates currently available techniques to determine (a) load-deformation characteristics of the various paving components that are suitable for use in performing a practical fatigue analysis and (b) the response in terms of stress, strain, and deflection of the pavement structure when subjected to traffic loading. Emphasis is placed throughout the paper on practical aspects of material characterization and structural analysis. An extensive list of references is given. The survey of literature indicates that suitable dynamic laboratory tests are available for use in characterizing pavement material properties in analytical or numerical solutions to layered pavement systems. A detailed discussion is given of material properties, testing techniques, and testing systems. Existing layered theories are reviewed, and a number of comparisons are given between calculated and measured responses of several layered pavement systems. A table summarizes the availability and limitations of several computer programs for predicting the response of layered pavement systems. Practical recommendations are given concerning the required laboratory testing and the layered theory suitable for use in a fatigue subsystem.

Material Characterization and Layered Theory for Use in Fatigue Analyses

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In recent years, highway engineers have shown considerable interest in developing a mechanistic basis for designing and evaluating pavement structures. Linear-elastic, non-linear-elastic, and viscoelastic layered system theories for use in predicting pavement response are all currently either in use or under development. A number of recent studies (1, 2, 5, 6, 11, 17, 24, 33, 34, 40, 49) indicate that the performance of a flexible pavement is closely related to the stresses, strains, and displacements calculated from layer pavement theories in which experimentally measured material properties are used.

Proper design and evaluation of pavement structures require consideration of many factors: environment, traffic, material properties, construction variables, maintenance variables, and economics. In the past, most widely used design methods for asphalt concrete pavements have involved empirical correlations of field performance with material properties measured in the laboratory or the field or both. Use of those methods in many instances has provided reasonably satisfactorily performing pavements; however, major difficulties arise when those methods are extrapolated beyond the conditions for which they were originally developed. Only an improved pavement design and evaluation procedure based on the mechanistic behavior of the pavements could be extended to new service conditions.

The state of the art is such that the technical knowledge necessary to develop a mechanistic pavement design procedure is now available. Unfortunately, a price must be paid to accomplish that goal. More sophisticated tests are required to characterize the pavement materials, and a computer is required to predict the pavement response. The additional cost to develop suitable design methods, retrain personnel, and purchase new testing equipment, however, will permit a better understanding of pavement behavior and lead to more efficient use of paving materials.

The purpose of this paper is to examine and evaluate currently available techniques for determining (a) load-deformation characteristics of the paving components that are suitable for use in performing a practical fatigue analysis and (b) stress, strain, and deflection responses of the pavement structure when it is subjected to traffic loadings. Rutting is not considered in this paper but has been treated in detail elsewhere (1, 4). Emphasis is placed throughout the paper on practical aspects. Specific recommendations are given concerning selection, use, and availability of laboratory testing equipment and computer programs that are suitable for application in a mechanistic pavement design method.

MATERIAL CHARACTERIZATION

Proper design of asphalt concrete pavement systems relies in part on a thorough understanding of the response of the component materials to loading. Two broad areas of material characterization must be considered in pavement design: (a) material parameters for use in establishing failure criteria and (b) load-deformation characteristics of each component for use in calculating the physical response of the system. It is not likely that a properly designed pavement would fail under a single-axle loading. As a result, the traditional concept of shear strength (1, 16), which would define failure under a single load, will not be discussed in this paper. Under a large number of repetitions, however, fatigue cracking of stabilized materials frequently occurs because of repeated bending. That is the mode of failure of concern in this symposium. The mechanism of fatigue failure and methods for predicting fatigue are given in other papers in this Special Report (14, 56).

Practical Considerations

The material properties used in any mechanistic design procedure should be evaluated in tests that simulate as closely as practical the stress conditions occurring under the action of a moving load. When a wheel load moves past an element of material located beneath the pavement system, the element is subjected to stress states similar to that shown in Figure 1 (3). Each element of material is subjected to a simultaneous buildup in both the major principal stress σ_1 and the minor principal stress σ_3. As those stresses build up, a rotation of the principal stress axis also occurs (Fig. 1). In addition to duplicating the stress or strain states, the test should also attempt to simulate the environmental conditions that exist in the field.

The CBR and the R-value, and to a somewhat lesser extent the static triaxial tests, do a relatively poor job of simulating the in situ stress conditions. The in situ stress can, however, be reasonably approximated by the use of either the repeated load triaxial test or the cyclic triaxial test. The exact duplication of the stress pulse appears to be somewhat less important in determining the dynamic modulus of elasticity than in determining the rutting characteristics of pavement materials; in fact, creep (5, 49, 52), relaxation, and free vibration tests (29) have all been used with reasonably good results to evaluate the modulus of elasticity of pavement materials.

Figure 2 (3) shows the typical haversine-shaped major principal (or vertical) compressive stress pulse to which an element of material beneath the pavement is subjected. Square, triangular, sinusoidal, and haversine-shaped pulses have all been used to simulate the actual in situ stress. Any of those wave forms should give a satisfactory approximation for estimating the elastic response provided care is taken in selecting the magnitude and duration of the pulse. For conventional flexible pavement structures and spring and summer temperatures, the duration of the stress pulse varies primarily with the location of the element with respect to the point of loading and with the velocity of the vehicle. The curves shown in Figure 3 are suggested as a practical guide to selecting the duration of the stress pulse that should be used in a dynamic laboratory test.

Other things that must be considered in the testing of pavement materials are the effects of previous stress history and loading stress path. Dehlen (17) and later Hicks (33) found that for moderate stress levels the elastic response of most subgrade soils, unstabilized granular materials, and asphalt concrete will become relatively constant after approximately 100 load repetitions. Those studies also indicated that a single test specimen could be used to characterize the non-linear-elastic response of granular materials. That can be accomplished by determining the elastic bounce at several different confining pressures or deviator stresses or both provided care is exercised to gradually increase the severity of the stress state.

Advanced Concepts

Elastic Materials

A general anisotropic, linear-elastic material can be modeled by the following linear stress-strain law (21):

$$
\begin{aligned}
\sigma_x &= C_{11}\epsilon_x + C_{12}\epsilon_y + C_{13}\epsilon_z + C_{14}\gamma_{xy} + C_{15}\gamma_{yz} + C_{16}\gamma_{zx} \\
\sigma_y &= C_{21}\epsilon_x + C_{22}\epsilon_y + C_{23}\epsilon_z + C_{24}\gamma_{xy} + C_{25}\gamma_{yz} + C_{26}\gamma_{zx} \\
&\vdots \\
\tau_{yz} &= C_{61}\epsilon_x + C_{62}\epsilon_y + C_{63}\epsilon_z + C_{64}\gamma_{xy} + C_{65}\gamma_{yz} + C_{66}\gamma_{zx}
\end{aligned}
\qquad (1)
$$

where

$\sigma_{x,y,z}$ = normal stress components,
ϵ_{ij} = normal strain components,
τ_{ij} = shear stress components,
γ_{ij} = shear strain components, and
C_{ij} = material constants.

From energy considerations, $C_{ij} = C_{ji}$; and, as a result, a general anisotropic linear material can be characterized by 21 independent elastic constants. For an isotropic linear-elastic material, however, it has been shown (21) that a material is completely characterized by 2 elastic constants that can be determined from material tests. Usually in pavement design, the modulus of elasticity E and Poisson's ratio ν are the 2 elastic constants evaluated in the laboratory and used in layered theory. Once any 2 elastic constants are known, all other elastic constants can be determined by the use of Table 1 (21).

The stress-strain relations for an isotropic linear material can be expressed as follows:

$$
\begin{aligned}
\epsilon_x &= \frac{1}{E}\left[\sigma_x - \nu(\sigma_y + \sigma_z)\right] & \gamma_{xy} &= \frac{2(1+\nu)}{E}\tau_{xy} \\
\epsilon_y &= \frac{1}{E}\left[\sigma_y - \nu(\sigma_z + \sigma_x)\right] & \gamma_{yz} &= \frac{2(1+\nu)}{E}\tau_{yz} \\
\epsilon_z &= \frac{1}{E}\left[\sigma_z - \nu(\sigma_x + \sigma_y)\right] & \gamma_{zx} &= \frac{2(1+\nu)}{E}\tau_{zx}
\end{aligned}
\qquad (2)
$$

The proper use of those equations is quite important and will be illustrated by means of a simple example. Consider an element of material that is subjected to 3 tensile principal stresses and has the material properties shown in Figure 4. From Eq. 2 the

Figure 1. Rotation of principal stress axis of element as vehicle moves over surface.

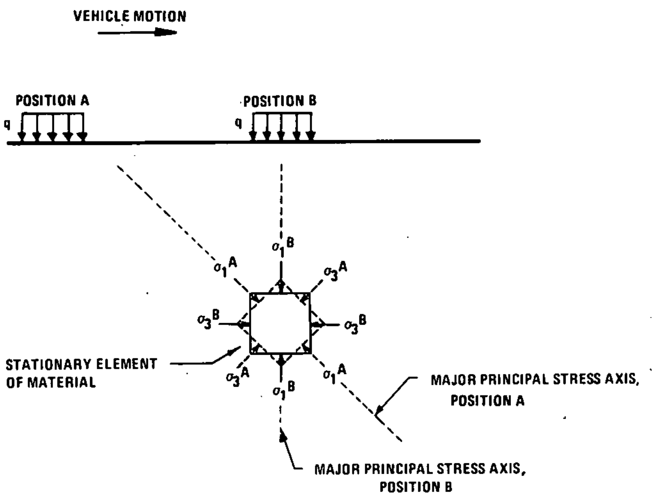


Figure 2. Equivalent sinusoidal and triangular stress pulses.

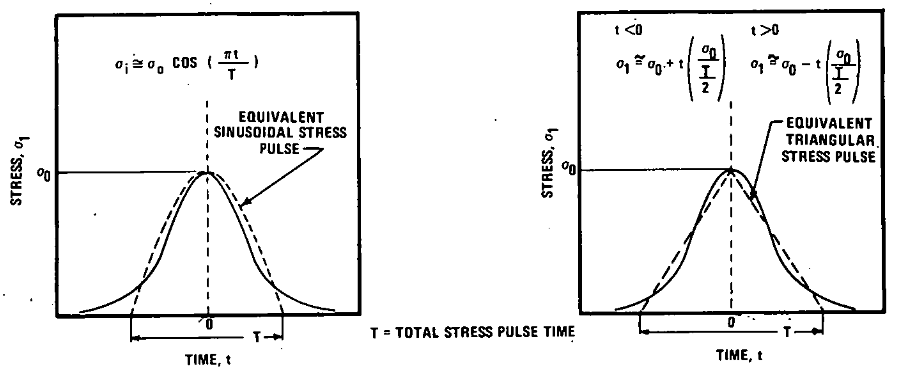
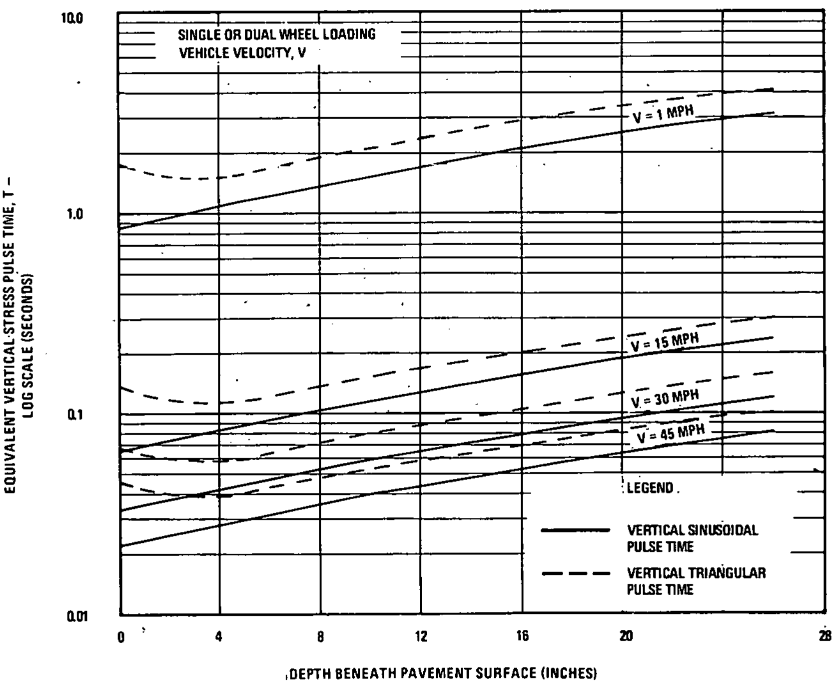


Figure 3. Variation of equivalent vertical stress pulse time with vehicle velocity and depth.



strain in the z-direction is then $\epsilon_z = (1/E)\,[\sigma_z - \nu(\sigma_x + \sigma_y)]$, or $\epsilon_z = (1/1{,}000)\,[10 - 0.4\,(30 + 20)] = -0.01$ in./in. (compression). That simple example readily illustrates that (a) $\epsilon_z \neq \sigma_z/E$ because of the effect of Poisson's ratio and (b) even though the stress in the z-direction is tensile the strain can be compressive also because of the effect of Poisson's ratio.

Viscoelastic Material Characterization

Both asphalt concrete and plastic clays exhibit strong viscous characteristics because their properties are greatly influenced by the frequency and duration of load. As a result, several investigators (5, 25, 26, 51, 52, 53) have quite logically characterized paving materials in both linear and nonlinear viscoelastic models. Usually, materials are assumed to be linearly viscoelastic to simplify their representation and make stress analyses practical. Whether a material is actually linearly viscoelastic must be determined experimentally.

The differential operator form of the linear viscoelastic stress-strain law (79) is most commonly used and can be visualized for conceptual purposes as a combination of springs and dashpots. A complete catalog of mechanical models, together with their stress-strain behavior for creep, relaxation, and constant strain-rate loadings, has been assembled by Williams, Blatz, and Schapery (78).

An isotropic, linear viscoelastic material can be represented by 2 independent linear viscoelastic operational moduli (material properties) such as $E(p)$ and $\nu(p)$, which are functions of the transform parameter p (transformed time). Since a direct analogy exists between the viscoelastic and elastic constants, the usual elastic formulas (Table 1) can be used to convert $E(p)$ and $\nu(p)$ to, for example, $G(p)$ and $K(p)$.

Laboratory Test Methods

Flexible pavement materials are to varying degrees nonhomogeneous, anisotropic, nonlinear, and nonelastic. Some of their properties are time dependent and affected by changes in the environment such as temperature or moisture content. A detailed list of variables affecting material response was previously reported by Deacon (16).

The elastic or viscoelastic material properties for use in a mechanistic pavement design procedure have been determined in several types of tests. Tests that have been used most frequently in the past and continue to be the most promising from a practical viewpoint are repeated-load triaxial, cyclic load triaxial, and creep. Other laboratory tests that either have been or could be employed to characterize materials for use in layered system theories include flexural bending, hollow cylinder, torsion, and indirect tension (5, 51, 52).

Evaluation of Elastic Constants

The repeated-load test has been most frequently used to evaluate the elastic constants E and ν of paving materials. In that test, a cylindrical specimen is placed in a conventional triaxial cell and subjected to repeated deviator stress pulses such as the rectified sinusoidal wave form shown in Figure 5a. A few researchers (17, 51, 52) have subjected the specimen to simultaneous repeated axial (Fig. 5a) and lateral stress states (Fig. 5b), which duplicate reasonably closely the stress conditions that exist in the field. Most tests, however, have used a constant cell pressure and are much simpler to perform. Careful selection of a constant confining pressure should give results that are satisfactory for use in a practical mechanistic approach. Additional work is now being conducted to determine the advantages of repeated confining pressure (8).

Resilient Modulus

The resilient modulus of elasticity E_R obtained from the repeated-load triaxial test is defined as the deviator (or repeated axial) stress divided by the recoverable strain

Table 1. Relations between elastic constants in isotropic linear materials.

Elastic Constant	λ Dimension FL^{-2}	G Dimension FL^{-2}	E Dimension FL^{-2}	ν Dimension 1	K' Dimension FL^{-2}
λ					
G	—	—	$\frac{G(3\lambda + 2G)}{\lambda + G}$	$\frac{\lambda}{2(\lambda + G)}$	$\frac{3\lambda + 2G}{3}$
E	—	$\frac{A^{*} + (E - 3\lambda)}{4}$	—	$\frac{A - (E + \lambda)}{4\lambda}$	$\frac{A + (3\lambda + E)}{6}$
ν	—	$\frac{\lambda(1 - 2\nu)}{2\nu}$	$\frac{\lambda(1 + \nu)(1 - 2\nu)}{\nu}$	—	$\frac{\lambda(1 + \nu)}{3\nu}$
K	—	$\frac{3(K - \lambda)}{2}$	$\frac{9K(K - \lambda)}{3K - \lambda}$	$\frac{\lambda}{3K - \lambda}$	
G					
E	$\frac{G(2G - E)}{E - 3G}$	—	—	$\frac{E - 2G}{2G}$	$\frac{GE}{3(3G - E)}$
ν	$\frac{2G\nu}{1 - 2\nu}$	—	$2G(1 + \nu)$	—	$\frac{2G(1 + \nu)}{3(1 - 2\nu)}$
K	$\frac{3K - 2G}{3}$	—	$\frac{9KG}{3K + G}$	$\frac{3K - 2G}{2(3K + G)}$	
E					
ν	$\frac{\nu E}{(1 + \nu)(1 - 2\nu)}$	$\frac{E}{2(1 + \nu)}$	—	—	$\frac{E}{3(1 - 2\nu)}$
K	$\frac{3K(3K - E)}{9K - E}$	$\frac{3EK}{9K - E}$	—	$\frac{3K - E}{6K}$	
ν					
K	$\frac{3K\nu}{1 + \nu}$	$\frac{3K(1 - 2\nu)}{2(1 + \nu)}$	$3K(1 - 2\nu)$	—	

$^{*}A = \sqrt{(E + \lambda)^2 + 8\lambda^2}$.

Figure 4. Properties of example material.

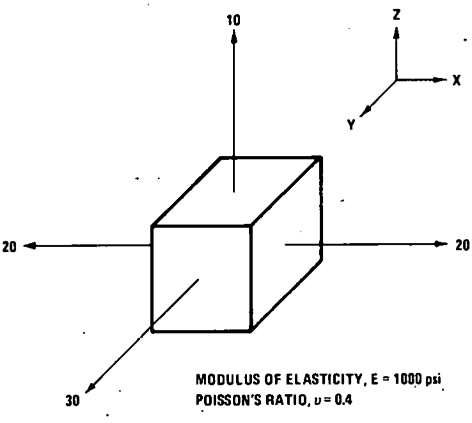
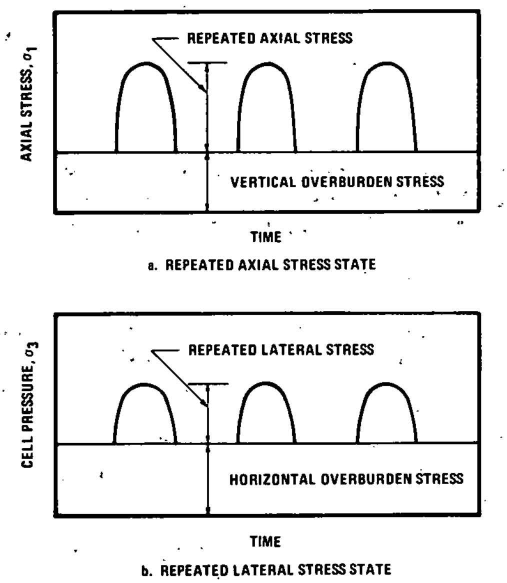


Figure 5. Stress states in repeated load triaxial tests using sinusoidal stress pulse.



associated with the bounce of the specimen. By definition, a secant modulus is obtained that corresponds to the minimum value occurring during the loading portion of the test.

In any dynamic tests, the deformation as well as loads must be measured by electronic measuring and recording equipment. Many times undesirable deformations can occur during loading in the piston and end platens and also in the associated connections between components. If those movements are included in the deformation used to calculate the recoverable strain, the calculated resilient modulus will be smaller than the actual value. Results indicate that when the resilient modulus is greater than about 15,000 psi special measuring devices should be used inside the cell to eliminate that problem. Reasonably reliable resilient deformation measurements can be obtained by attaching 2 thin, circular aluminum or plexiglass clamps around the specimen approximately at the quarter points (Fig. 6). Theoretical studies (17) indicate that the stiffening effect of the clamp should not increase the resilient modulus by more than about 10 to 15 percent.

Reliable axial deformations of the specimens can be obtained by the use of 2 linear variable differential transducers (LVDT) attached to the clamps, placed on opposite sides of the specimen, and wired so that their electrical outputs are added together (or averaged) and then recorded on a reasonably fast-responding electronic recorder. At the Georgia Institute of Technology deformations are usually measured with a dc electrical system by means of a pair of Collins SS-203 or SS-204 LVDT's. Measurements can also be obtained by ac recording systems. The advantage of using an ac measuring system is that the LVDT's are lightweight and cost only about a third as much as dc LVDT's. An ac recording system should not, however, be used without suitable correction networks when phase angle relations are to be measured between stress and strain. Of course, many other types of measuring systems, such as displacement potentiometers and optical scanners, can be successfully used.

Wire resistance strain gauges bonded directly on the specimen provide an excellent means for directly measuring the resilient strain in stabilized materials such as asphalt concrete and cement-stabilized materials. The active strain gauge length should probably be equal to or greater than twice the maximum diameter of the aggregate in order to measure the average strain in those specimens.

Poisson's Ratio

Poisson's ratio is an elastic constant that is difficult to reliably evaluate for most pavement materials. For an ideal, isotropic, cylindrical specimen of material subjected to a uniform principal stress state, Poisson's ratio is equal to

$$\nu = -\epsilon_l/\epsilon_a \tag{3}$$

where ϵ_l and ϵ_a are the lateral and axial strains respectively. Dehlen (17) has theoretically shown that, if perfectly frictionless caps and bases are used, the errors associated with uneven lateral strain of the specimen should be less than 10 percent. Therefore, Eq. 3 can be used to calculate Poisson's ratio provided that end friction is eliminated.

From physical considerations, ν for elastic isotropic materials should be between -1 and $\frac{1}{2}$. However, experimentally determined values of ν from the repeated-load test are in some instances greater than $\frac{1}{2}$ (33). Those large values of ν may at least partially be caused by the nonuniform stress and deformation conditions that exist in the triaxial specimens and also by pavement materials not behaving as ideal elastic solids.

Most researchers (1, 17, 24, 33, 37) who have attempted measurements of ν have used either wire resistance strain gauges for stabilized materials or LVDT's for nonstabilized materials. For bound materials (1, 17, 33), a pair of strain gauges can be bonded to the specimen at midheight with the gauge oriented horizontally (17, 33). Lateral deformation in clay or unbound gravel or both has been measured by 2 ac transducers fixed to aluminum or plexiglass clamps at the quarter points (33). For clays, Dehlen (17) also drilled diametral holes through the sample and used an LVDT to measure the lateral deformation. Another approach to measure lateral deformation is by the use of a lateral deflectometer (Fig. 7, 5). That consists of 3 thin metal probes that press against the specimen and are supported on an aluminum ring positioned about the center of the specimen. A strain gauge is bonded to the side of each probe to measure the strain in it as the specimen deforms.

Poisson's ratio can also be determined from the total volume change that the specimen undergoes. From the theory of elasticity, Poisson's ratio is related to the volume change by the following approximate relation:

$$\nu \simeq \frac{1}{2}\left(1 - \frac{1}{\epsilon_a}\frac{\Delta V}{V}\right) \tag{4}$$

where

ν = Poisson's ratio,
ΔV = change in volume of the specimen,
V = initial volume of the specimen, and
ϵ_a = axial strain.

The volume change can be evaluated by measuring the deformation profile of the specimen directly or by filling the cell with a fluid and measuring its change in volume (83).

Evaluation of Viscoelastic Properties

Viscoelastic material properties are usually determined experimentally by creep, relaxation, and sinusoidal stress input or sinusoidal strain input tests (5, 54). Athough low order spring and dashpot models have sometimes been used to characterize test results, they cannot realistically represent experimental data during more than 1 or 2 log cycles of time.

Probably the best method available at this time to evaluate the dynamic modulus of the asphalt concrete surfacing is to apply a sinusoidally oscillating stress. That method, which is referred to in this paper as the cyclic triaxial test, minimizes impact effects that may become important as the equivalent frequency of loading becomes relatively large as, for example, in the surfacing when a vehicle exceeds a speed of 30 to 40 mph. The dynamic modulus $|E^*|$, which can also be used in an elastic analysis, is equal to the peak sinusoidal stress σ_o divided by the peak recoverable axial strain ϵ_o (Fig. 8, 12).

A good alternative approach, which does not require expensive test equipment, is to use a creep test to evaluate the dynamic elastic or viscoelastic modulus of pavement materials. If that approach is used, the sample should probably be carefully preconditioned by methods such as statically cycling the axial load through about 4 to 6 repetitions. A conventional triaxial cell can be used together with a simple loading arrangement (12, 51, 53). Comparative studies should, however, be made if possible between the dynamic modulus obtained from the creep test and either the repeated-load or the cyclic triaxial test.

Descriptions of techniques for handling the viscoelastic properties are described elsewhere (2, 5, 59, 60, 79). From either creep or sinusoidal test results, a dynamic viscoelastic modulus for any loading frequency can be predicted from the results of a single creep or dynamic test by the use of a generalized Kelvin model (70). Coffman et al. (12) and Kallas (37) have both used this approach to evaluate the dynamic elastic properties of pavement materials. Typical values for $|E^*|$ determined by this method are shown in Figure 9 (37) for materials having 5 percent asphalt and 3.8 percent air voids.

Testing Equipment

Dynamic properties of pavement materials can be evaluated by the use of a mechanical system (12, 42), a pneumatic system (64, 65), an open-loop hydraulic system (20), and a closed-loop hydraulic servosystem (37). The advantages, disadvantages, approximate costs, and selected sources of those testing systems are as given below.

Figure 6. Circumferential clamp and LVDT for measuring axial deformation under repeated loading.

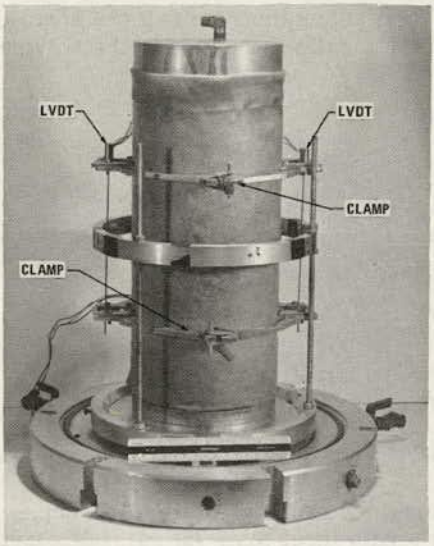


Figure 7. Diameter deflectometer positioned to measure change in diameter at center of granular base specimen.

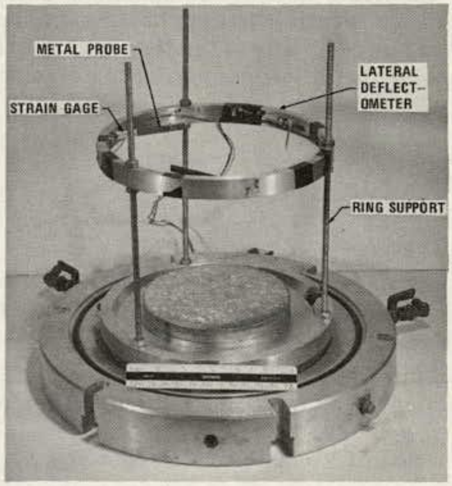
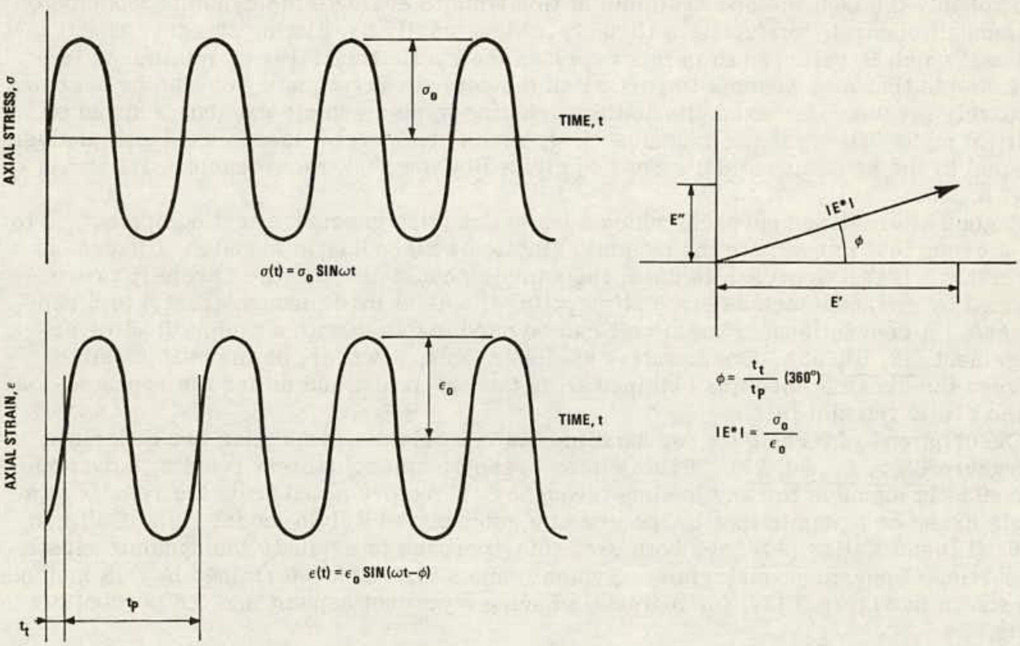


Figure 8. Dynamic test recording.



1. The mechanical system is relatively reliable but has some problems with design, balance, and operation. It can apply desired load pulse shapes by adjusting cams. Its maximum response is between 5 and 25 Hz, and its cost is between $4,000 and $6,000. It is available by special fabrication only.

2. The pneumatic system is relatively simple, cheap, reliable, and easy to design and repair. It will require periodic replacement of valves and cylinder. Its practical load limit is about 3,000 to 5,000 lb. An exact pulse shape is hard to apply. Its maximum response is between 5 and 8 Hz, and its cost is between $3,000 and $5,000. Selected sources are Geotechnical Research, Inc., 2403 Wylie Drive, S.E., Marietta, Georgia 30062; Research Engineering, 2640 Dundee Road, San Pablo, California; and Structural Behavior Engineering Laboratory, Inc., Post Office Box 9727, Phoenix, Arizona.

3. The hydraulic or open-loop control system is more complex to design and set up than the pneumatic system and requires a hydraulic pump and storage reservoir. A hydraulic system has a faster response (5 to 8 Hz) than a pneumatic one and can go to much higher loads. An exact pulse shape is hard to apply. It costs between $4,500 and $8,000 and is available by special fabrication only.

4. The hydraulic servo or closed-loop control system can have a fast response (25 to 100 Hz), high load capacity, and capability to apply any pulse shape to specimens. Disadvantages are its high initial cost (between $10,000 and $30,000) and maintenance cost and complex electronics. Some systems are very hard to balance and keep in proper operating condition. Selected sources are Hydratech, 2890 John Road, Troy, Michigan 48084; Mb Electronics, Post Office Box 1825, New Haven, Connecticut 06508; and MTS Systems Corporation, Minneapolis, Minnesota 55424.

For routine testing of pavement materials, a reliable system that is easy to maintain and repair is essential. Furthermore, if the dynamic material properties of all layers of a flexible pavement are to be evaluated, the system should have as minimum requirements a load capacity of at least 1,500 to 2,000 lb and the capability of applying a pulse to the specimen in 0.1 sec or less and at frequencies ranging from approximately 0.5 to at least 5 Hz.

The closed-loop hydraulic servosystems have by far the best overall capability. Those systems, however, can be "electronic monsters"; they are often quite expensive and time-consuming to maintain properly for even routine operations. As a result they are not considered to be suitable as a production type of testing system for use in most highway materials laboratories. The pneumatic testing system (or a slightly faster acting air-oil system) does not have nearly the overall capability as that of a closed-loop testing system. However, if properly designed, it can meet the minimum requirements for the dynamic testing of pavement materials and is very reliable and easy to maintain. Because of its relatively low cost and high degree of reliability, the pneumatic (or air-oil) type of system is recommended for routine dynamic testing. Where loads of more than 4,000 to 5,000 lb are required, such as for loading prototype pavement systems, an open-loop hydraulic system can often be used to good advantage.

Dynamic Properties of Pavement Materials

Results of dynamic tests on all materials show that the dynamic modulus, and to some extent Poisson's ratio, depends on the stress state. Because of nonlinearity, serious errors can arise if E is not evaluated by the use of a stress state that is compatible with the one that will exist in the pavement.

Typical values for the dynamic modulus of elasticity of various materials in the pavement section are given in Table 2. Those values can be used in the design of pavements in the absence of actual laboratory test results but should be considered as approximate and used with considerable caution.

Cohesive Subgrade Soils

In general, the resilient modulus E_R of cohesive soils decreases with increasing repeated stress level σ_d (Fig. 10, 70) and for some soil types is relatively unaffected

Figure 9. Dynamic modulus of asphalt concrete as function of loading frequency.

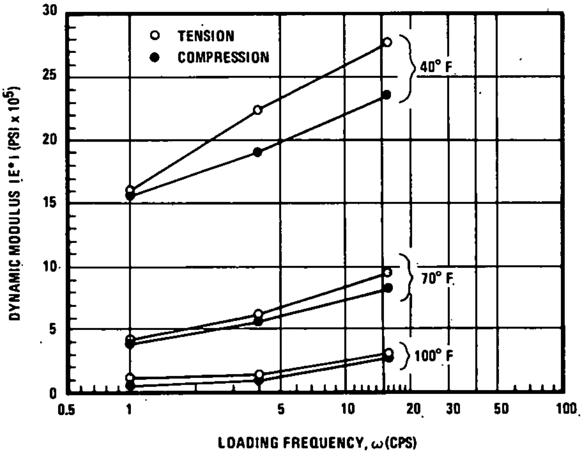


Table 2. Selected measured dynamic moduli for pavement materials.

Reference of Test Method[a]	Material Description	Frequency and Duration	Load Repetition	Dynamic Modulus
Asphalt Concrete				
33	California type B; ½ in. max med. aggr.; 85 to 100 pen. asphalt	30 cpm, 0.1 sec	100	300,000 psi, 70 F 70,000 psi, 90 F
1	Georgia standard A; 1½ in. max aggr.; 85 to 100 pen. asphalt	20 cpm, 0.1 sec	10,000	220,000 psi, 72 F 100,000 psi, 89 F
51	California type B; ⅜ in. max med. aggr.; 85 to 100 pen. asphalt	30 cpm, 0.1 sec	100	2,500,000 psi, 40 F 1,500,000 psi, 55 F 500,000 psi, 70 F 50,000 psi, 100 F
37	Asphalt Institute mix IVb; ½ in. max aggr.; 60 to 70 and 85 to 100 pen. asphalts	1 to 16 cps	250 to 300	600,000 to 2,000,000 psi, 40 F, 1 cps 150,000 to 750,000 psi, 70 F, 1 cps 50,000 to 150,000 psi, 100 F, 1 cps 1,100,000 to 3,000,000 psi, 40 F, 16 cps 350,000 to 1,300,000 psi, 70 F, 16 cps 90,000 to 450,000 psi, 100 F, 16 cps
Unbound Granular Base				
36	Colorado; standard base, ½ in. max and 8.7 percent < No. 200; standard subbase, 2½ in. max and 7.9 percent < No. 200	120 cpm, 0.2 sec	10,000	10,618 psi, σ_3 0.447, 2.4 percent w-c 10,019 psi, σ_3 0.465, 6.3 percent w-c 8,687 psi, σ_3 0.496, 8.2 percent w-c
33	California; well-graded and sub-rounded gravel ¾ in. max; class 2 aggr. base	30 cpm, 0.1 sec	100	Dry 10,000 to 13,000 psi, 3 percent < No. 200, σ_3 0.53 8,000 to 9,000 psi, 8 percent < No. 200, σ_3 0.59 Partially saturated 7,000 to 10,000 psi, 3 percent < No. 200, σ_3 0.55 5,000 to 7,000 psi, 8 percent < No. 200, σ_3 0.60
33	California; well-graded and angular crushed stone ¾ in. max; class 2 aggr. base	30 cpm, 0.1 sec	100	Dry 11,000 to 12,000 psi, 3 percent < No. 200, σ_3 0.57 14,000 to 15,000 psi, 10 percent < No. 200, σ_3 0.50 Partially saturated 9,000 to 10,000 psi, 3 percent < No. 200, σ_3 0.57 7,500 to 9,500 psi, 10 percent < No. 200, σ_3 0.57
1	Soil-aggregate of 17 percent silty sand and 83 percent crushed granite; 100 percent T-180; w-c = 5.1 percent	33 cpm, 0.1 sec	10,000	3,836 psi, $\theta^{0.53}$ 3,145 psi, $\theta^{0.55}$
65	California; well-graded and sub-rounded gravel ¾ in. max; class 2 aggr. base (dry)	30 cpm, 0.2 sec	10,000	7,000 psi, σ_3 0.55
1	Silty fine sand 100 percent AASHO T-99; 40 percent < No. 200, w-c = 13.4 percent	33 cpm, 0.1 sec	10,000	1,856 psi, $\theta^{0.61}$ 3,126 psi, $\theta^{0.37}$
Subgrade				
36	AASHO class A-6 silty clay; w-c = 14 to 18 percent; γ_d = 110 to 114 pcf	120 cpm, 0.2 sec	10,000	3,000 to 4,000 psi, 18 percent w-c 7,000 to 8,000 psi, 16 percent w-c 15,000 to 20,000 psi, 14 percent w-c
1	Micaceous silty sand subgrade	33 cpm, 0.1 sec	10,000	3,000 to 4,000 psi, wet season 1,500 to 2,000 psi, dry season
64	Silty clay (AASHO test); σ_d = 5 to 10 psi; σ_3 < 3 psi; γ_d = 110 to 115 pcf	20 cpm, 0.25 sec		13,000 psi, 13 percent w-c 10,000 psi, 14 percent w-c 8,000 psi, 15 percent w-c 7,000 psi, 16 percent w-c 2,000 to 5,000 psi, 17 percent w-c 2,000 psi, 18 percent w-c
65	Highly plastic clay (PI = 36.5) and silty clay (PI = 25.5)	30 cpm, 0.1 sec	10,000	4,150 psi, σ_d 1.0 3,200 psi, σ_d 5.2
36	AASHO class A-7-6 silty clay; w-c = 11 to 20 percent; γ_d = 102 to 105 pcf	120 cpm, 0.2 sec	10,000	7,000 to 10,000 psi, 20 percent w-c 15,000 to 16,000 psi, 18 percent w-c 14,000 to 15,000 psi, 16 percent w-c
17	AASHO class A-6 to A-7-6 silty clay	30 cpm, 0.1 sec	100	10,000 psi, 1 atm[b] 100,000 psi, 10 atm

[a]All are repeated load triaxial except reference 37, which is cyclic load triaxial. [b]Refers to soil moisture suction at time of test.

by small changes in confining pressure. The effect of confining pressure on the modulus appears to become greater as the clay decreases and the material becomes stiffer. The simplified nonlinear model also shown in Figure 10 has been used for clay subgrade soils where K_1, K_2, K_3, and K_4 are laboratory-determined constants.

The variation of ν with stress level is less clear although Hicks and Finn (34) found that it remained constant or increased slightly with increasing repeated vertical stress. Poisson's ratio, however, appears not to be significantly affected by confining stress.

Unstabilized Granular Base

For unstabilized granular materials, both E_R and ν are functions of the applied stress conditions. Numerous studies (1, 20, 33, 55, 65) of the resilient response of sands, gravels, and crushed stone have indicated that E_R significantly increases with confining pressure and is affected to a much smaller extent by the magnitude of the repeated vertical stress. As shown in Figure 11 (33), the resilient modulus of granular materials (partially crushed aggregate, low density, coarse grading) can usually be approximated by

$$E_R = \bar{k}\theta^{\bar{n}} \tag{5}$$

where $\bar{k}$ and $\bar{n}$ are constants evaluated from repeated-load triaxial test results and θ is the sum of the principal stresses ($\theta = \sigma_1 + 2\sigma_3$ in a conventional triaxial test). Studies have also shown that an expression of the form $E_R = k\sigma_3^n$ can also be used as long as the variation in the deviator stress is not too great for a given confining pressure.

Poisson's ratio of granular materials can vary considerably with the stress state and is apparently related to the principal stress ratio σ_1/σ_3 (33). The effects on E_R and ν of aggregate type, water content, density, and gradation have been described elsewhere (1, 33).

Asphalt-Bound Materials

Results of repeated-load triaxial tests have indicated that the temperature and rate (or frequency) of loading have a very significant effect on the stiffness of asphalt-bound materials (Fig. 9); asphalt content and type, air voids, and aggregate grading and type—all have a lesser effect on the stiffness. Several studies (18, 51, 55) found that the stiffness is stress dependent (i.e., the material is nonlinear), but that effect is still relatively small compared to that of temperature. Results indicate that Poisson's ratio may undergo an important increase with increasing temperature and to a lesser extent is affected by the stress level (Fig. 12, 51).

Although the results from flexural and indirect tensile tests show the same general trends, E_R determined in those tests is usually less than the value determined in the triaxial tests by as much as a half. Probably a value of E_R somewhere between those 2 extremes is appropriate to use for pavement design.

Cement-Bound Materials

The results of repeated-load tests on cylindrical specimens performed by Mitchell and Chen (44), Wang (85), and Barksdale (1) have shown that the E_R of soil-cement decreases with increasing confining pressure. Laboratory test results indicate that the resilient modulus measured in the triaxial test may be as much as 10 times greater than that obtained in repeated flexure (55), although one researcher (85) has shown results indicating the flexural modulus to be greater. The results of the flexure tests should probably be used in layered theory because that test more closely simulates conditions of bending, which occurs in very stiff base layers. Also, because of cracking with time in cement-treated bases, the effective modulus will tend to decrease significantly.

From the results of repeated-load triaxial tests on a cement-stabilized base, Fossberg (24) has shown that E_R is not very stress dependent and is essentially equal in

Figure 10. Effect of deviator stress on resilient modulus of fine-grained soil.

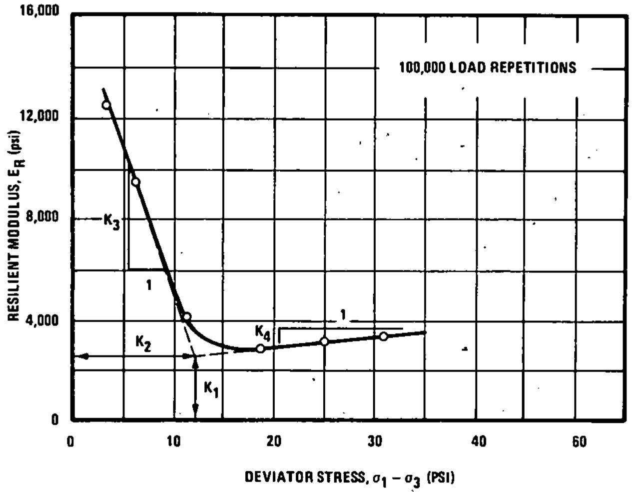


Figure 11. Variation in modulus with sum of principal stresses.

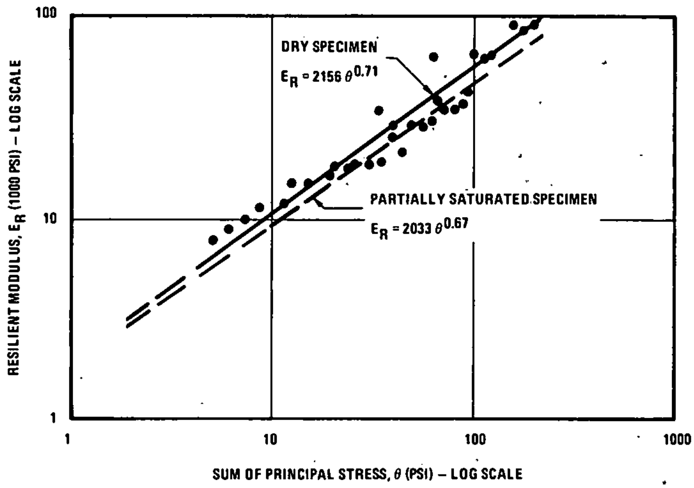
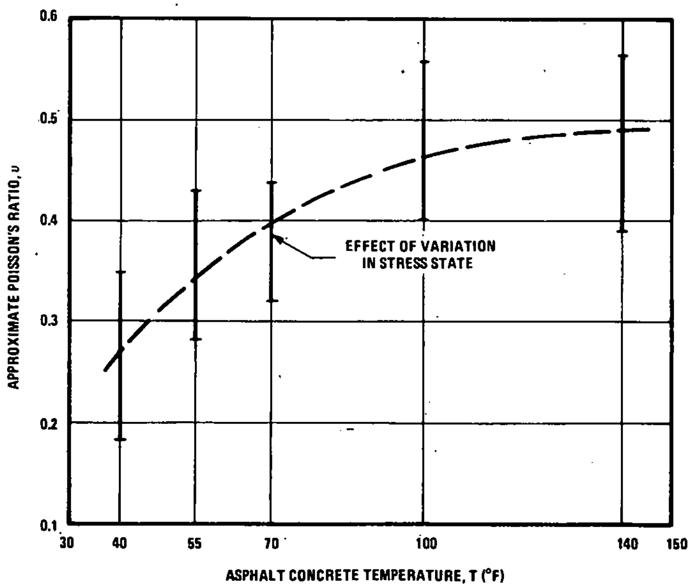


Figure 12. Approximate variation of Poisson's ratio with temperature.



tension or compression so long as cracking does not occur. Poisson's ratio was found to usually be between the limits of 0.1 and 0.2.

Empirical Correlations

In some instances it may become necessary to estimate the dynamic modulus of elasticity of pavement materials from empirical correlations with more easily measured material properties. Several such correlations have been developed by researchers for estimating the dynamic modulus of elasticity of pavement materials, but none has been developed for estimating Poisson's ratio.

Asphalt-Bound Materials

If direct measurements of stiffness are not possible, estimates may be made by the use of nomographs developed by Van der Poel (72) or Heukelom and Klomp (31) to obtain the stiffness of the bitumen as a function of time of loading and temperature (Fig. 13, 72). After the stiffness of the bitumen has been determined, the following empirical relations can be used to obtain the stiffness of the asphalt mixture:

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

where

S_{mix} = mixture stiffness, kg/cm^2;
S_{bit} = bitumen stiffness, kg/cm^2;
C_v = volume of aggregate/(volume of aggregate + volume of asphalt); and
n = $0.83 \log(4 \times 10^5)/S_{bit}$.

The following correction is normally applied to mixes having void contents larger than 3 percent:

$$C_v' = \frac{C_v}{1 + \Delta H}$$

where

C_v' = modified volume concentration of aggregate,
C_v = original volume concentration of aggregate, and
ΔH = difference between air void content and 3 percent, expressed as decimal.

Fair agreement (less than 50 percent error) has been shown between the nomograph and measured value of dynamic stiffness (48).

Soil and Unbound Aggregate

A limited number of correlations have been made between the dynamic modulus of elasticity of soils and unbound base materials with routine test procedures. Heukelom and Klomp (31), for unstabilized clay and sandy soils, have proposed a relation between the dynamic subgrade modulus of elasticity and the familiar CBR value as follows:

$$E(\text{psi}) = \text{constant} \times \text{CBR} \tag{6}$$

Normally a value of E(psi) = 1,500 CBR has been used for design purposes although the constant has been shown to vary from 700 to 10,000. Kirnan and Glynn (86) developed similar relations for 2 boulder clays:

$$E(\text{psi}) = 250 \times \text{CBR} \tag{7}$$

The tremendous variations in those correlations clearly indicate the magnitude of error that can result from using such relations.

Other more generalized correlations for estimating the dynamic modulus of elasticity for the unstabilized base and subgrade materials have been summarized by Shell (87) and are shown in Figure 14. In addition, for granular materials, E_R can be crudely estimated from a knowledge of the supporting layers. Heukelom and Klomp (31) observed on the basis of field measurements that E_R of the granular layer is approximately 2 times greater than that of the supporting layer. That multiplying factor, however, can vary from less than 1 to approximately 5; the lower values appear to be associated with a dry, stiff subgrade.

Limitations

Great caution should be exercised in using generalized empirical relations for estimating the dynamic modulus because they were developed for a very limited number of soil types and conditions; even then, the scatter in test results was appreciable. For one micaceous silty sand subgrade soil, the empirical correlation given in Eq. 6 indicated a dynamic modulus of 4,500 to 6,000 psi, whereas the actual resilient modulus was found to be only 2,000 psi. It is quite possible that use of generalized empirical correlations together with a mechanistic design approach could give results that have the same limitations as those associated with currently used empirical design methods.

Practical Application

The dynamic elastic properties of pavement materials can be evaluated by the use of either the repeated-load or the cyclic triaxial test. For routine production work, the repeated-load test using a pneumatic or air-oil loading system (or a mechanical system applying a sinusoidal loading) is the most practical approach for evaluating the dynamic modulus and should be used by most highway department material laboratories.

Under field service conditions, variation in the dynamic modulus due to changes in environmental factors such as moisture content and temperature can have significant effects on the overall performance and life of the pavement. For example, a recent study of pavements in Georgia (1) indicates that approximately 75 percent of the fatigue damage to pavements occurs during the wet season of the year. Therefore, even though reasonably accurate values of the dynamic modulus of pavement materials may be obtained in the laboratory, extreme care must be exercised in incorporating those data into mechanistic design procedures. To obtain the resilient modulus of asphalt-bound layers requires that estimates be made of the variation of temperature with depth, time of day, and season. Theoretical methods of predicting temperature are now available and give reasonably good results (38).

Another difficult problem is defining the in situ degree of saturation for different soils, drainage conditions, and groundwater table locations. A good alternative approach for estimating the degree of saturation is to develop generalized procedures using the soil suction profile (88).

Correlations can be made between the resilient modulus and carefully selected pertinent tests for specific soil types and conditions. For fine-grained silt and clay subgrade soils, correlations could be made, for example, between E_R and soil suction (17). Similar correlations could be developed by the use of creep tests or possibly free vibration tests for all types of materials. Some evidence indicates that E_R does not vary greatly for unstabilized granular bases so long as the geologic source, grading, relative density, and moisture content of the materials remains about the same. Under those conditions, a good estimate of E_R can in many instances be made by the use of previously determined laboratory relations such as $E_R = \bar{k}\theta^{\bar{n}}$.

Because of the problems associated with measuring accurate values of ν as well as the fact that the response of the pavement is relatively insensitive to reasonable variations, estimated values of ν can be used as an engineering approximation for the mechanistic design procedures. For asphalt concrete mixes, Poisson's ratio appears to vary between approximately 0.25 and 0.35; for unstabilized granular subbases and bases, $0.30 \le \nu \le 0.4$; and for clayey subgrades, $0.4 \le \nu \le 0.5$. A limited number of tests could, of course, be performed to establish representative values of ν for specific materials and structural conditions.

Figure 13. Nomograph for determining stiffness of bitumens.

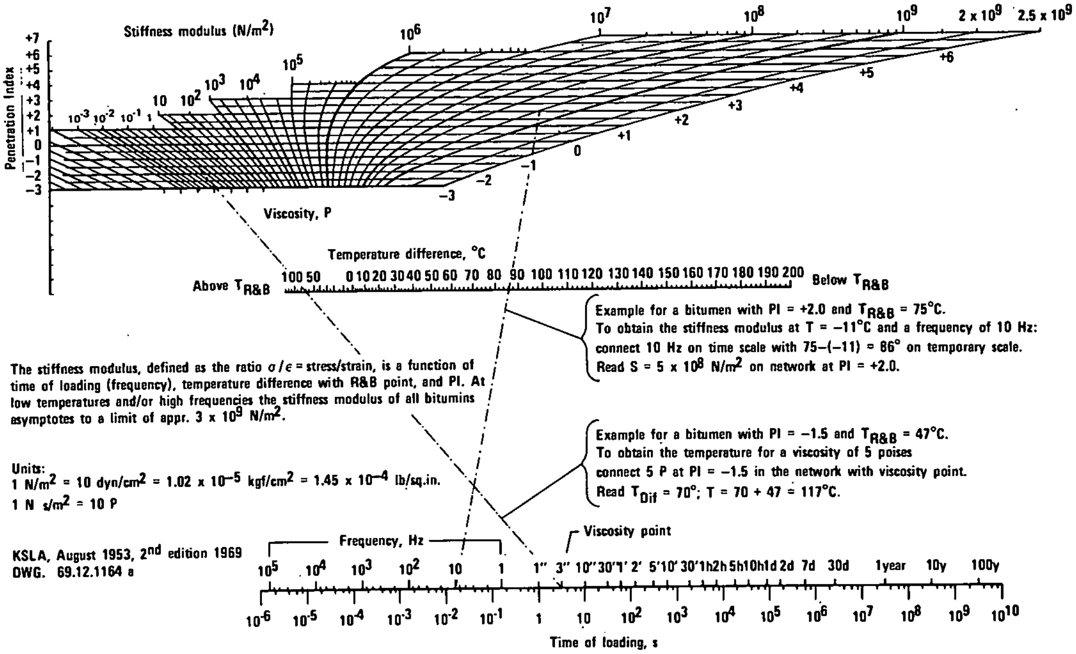
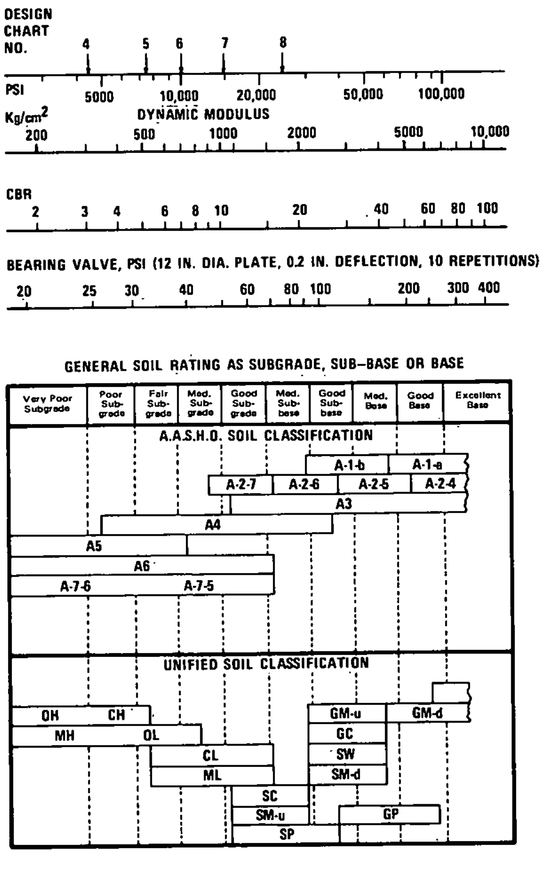


Figure 14. Crude empirical relations between dynamic modulus of elasticity and routine tests.



LAYERED SYSTEM ANALYSIS

Procedures for the prediction of traffic-induced deflections, stresses, and strains in pavement systems are based on the principal of continuum mechanics. The essential factors that must be considered in predicting the response of layered pavement systems are (a) the stress-strain behavior of the materials, (b) the initial and boundary conditions of the problem, and (c) the partial differential equations that govern the problem. Fortunately the highway engineer need only concern himself with the stress-strain behavior of the material, the physical configuration of the problem, and the general assumptions that have been made or implied in developing solutions to the layered system problem.

Reasonably good predictions of pavement response to load can be obtained provided carefully selected material properties are used with theories that make realistic assumptions. Unfortunately, the solution of the pavement system problem requires the use of a high-speed digital computer. If an engineer selects a formula that is not applicable to his set of conditions, an incorrect answer is obtained; likewise, if a computer program not suited to the particular problem is used, equally poor results are obtained. Therefore, to properly use the theoretical solutions that are now available, an engineer must thoroughly understand the assumptions and limitations associated with the use of these methods.

Solutions for Layer Systems

Elastic Layered Systems

The response of pavement systems to wheel loadings has been of interest since 1926 when Westergaard (77) used elastic layered theory to predict the response of rigid pavements. Later Burmister (9) solved the problem of elastic multilayered pavement structures (Fig. 15) using classical theory of elasticity. The assumptions that Burmister and most others (57, 76) have made in developing closed-formed solutions are as follows:

1. Each layer acts as a continuous, isotropic, homogeneous, linearly elastic medium infinite in horizontal extent;
2. The surface loading can be represented by a uniformly distributed vertical stress acting over a circular area;
3. The interface conditions between layers can be represented as being either perfectly smooth or perfectly rough;
4. Each layer is continuously supported by the layer beneath;
5. Inertial forces are negligible;
6. Deformations throughout the system are small; and
7. Temperature effects are neglected.

The partial differential equations associated with that boundary value problem can be solved by the use of integral transforms (57, 76). The response is then obtained in the form of infinite integrals that must be numerically integrated. If a sufficiently close integration interval spacing is not used, or if the integration is not taken out far enough before "chopping" it off, convergence of the integral to the correct value may not occur.

Comprehensive tables and charts of influence values for 2- and 3-layer systems subject to uniform circular loadings are given in the literature (35, 54). The use of those tables and charts can be quite tedious and time-consuming for 3-layer pavement systems, and tabulated solutions for 4-layer systems are not practical. Therefore, for general pavement design applications, the use of a computer is a necessity from a practical standpoint.

Schiffman (67) and Westmann (90) have further generalized the Burmister theory to include either vertical or horizontal asymmetric surface loading for multilayered elastic systems. Computer programs for asymmetric loading conditions, however, are not available at the present time.

Finite-Element Approaches

The continuum mechanics approaches described in the previous section satisfy exactly the governing differential equations associated with the pavement design problem. That closed-form solution can only be obtained for the case of homogeneous, isotropic, and elastic layers. The finite-element method, on the other hand, offers the capability of modeling pavements in a considerably more realistic manner. In contrast to the closed-form analytical solutions, each element in the system can be given independent anisotropic material properties, and the layers need not be infinite in width. In addition, solutions of the finite-element formulation for displacements, stresses, and strains are obtained for each element of the grid. In closed-form solutions, those quantities must be calculated individually at each desired point. The finite-element approach, however, is a numerical approximation, and as a result the cost of computer time to solve problems by that method may be as much as 2 to 5 times that for classical elastic layered solutions.

Detailed descriptions of the finite-element method have been presented by Zienkiewicz and Cheung (91). Usually the pavement system problem is approximated as one of axial symmetry, and a cylindrical coordinate system represented by r, θ, and z is used. A layered pavement structure may be idealized as an assemblage of finite numbers of discrete triangular or rectangular ring-shaped elements (Fig. 16, 3). Each adjacent element in the system comes together at a common point and is interconnected by frictionless pins called nodes.

The structural stiffness properties of each element can be determined by the application of energy principles (91) using an approximate displacement function and the usual elastic relations for displacements, strains, and stresses. The selection of the displacement function is quite important and should be chosen so as to maintain internal displacement continuity inside each element. In general, interelement displacement compatibility should also be maintained so that gaps will not open up between adjacent elements. Finally, rigid body displacement states and all uniform strain states must also be included in the element characterization. Those element displacement requirements will then ensure a monotonic convergence of the strain energy function if the element mesh size used in the structural model is repeatedly divided. If a fine enough mesh of elements is used, the approximate finite-element theory will usually give a satisfactory solution. Convergence to the correct solution with decreasing mesh size is not, however, always guaranteed.

Limitations of Layered Theory

In both classical and finite-element layered theories, the pavement structure is normally modeled as an axisymmetric solid. Axisymmetry usually means that both load and pavement geometrics are symmetrical about a common centerline. Unfortunately, the effects of wheel loads applied close to a crack or pavement edge cannot be analyzed by the use of methods that require axisymmetry. Although 3-dimensional solid models could be used with the finite-element method, that representation is not very practical for general use because of the large amount of computer time required to solve the model. An extended 2-dimensional finite-element program that approximates the loading as a Fourier series has been used to study the effects of edge loadings for multiple rectangular wheel loadings (89). Although that approach should lead to a much better understanding of pavement behavior, it requires too much computer time for general use in a design method. Caution should also be exercised in using Fourier series expansions for the loading to ensure that a sufficient number of terms are used to give accurate results. Furthermore, information is not available on the condition of slip that exists at the interface between layers. The assumption of a rough interface condition, which most investigators have used, appears to be reasonable, although varying degrees of slip can be considered.

In all of the theoretical approaches, inertial forces have been neglected. The inertial force is simply the force on a small element caused by a dynamic loading equal to the mass of the element times the acceleration. Also, none of the layered system

Figure 15. Classical linear-elastic layered pavement structure idealization.

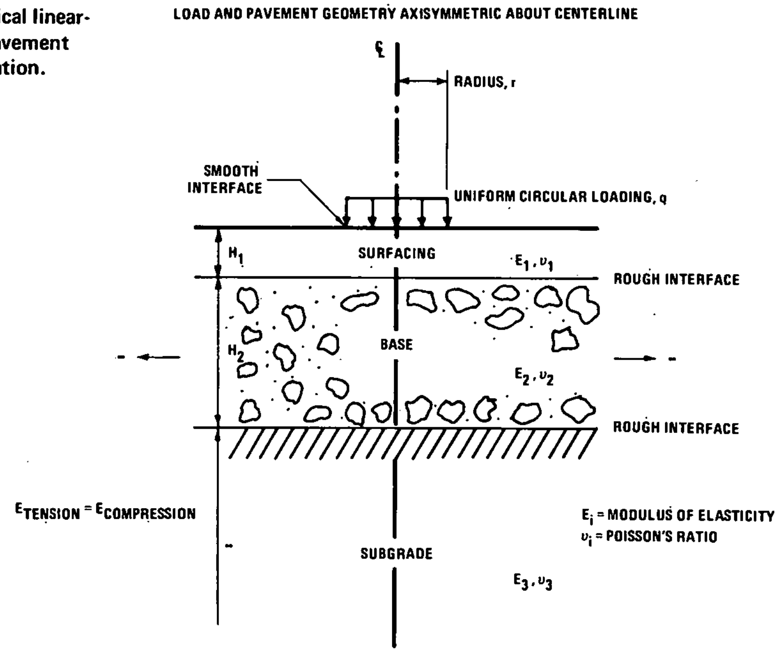
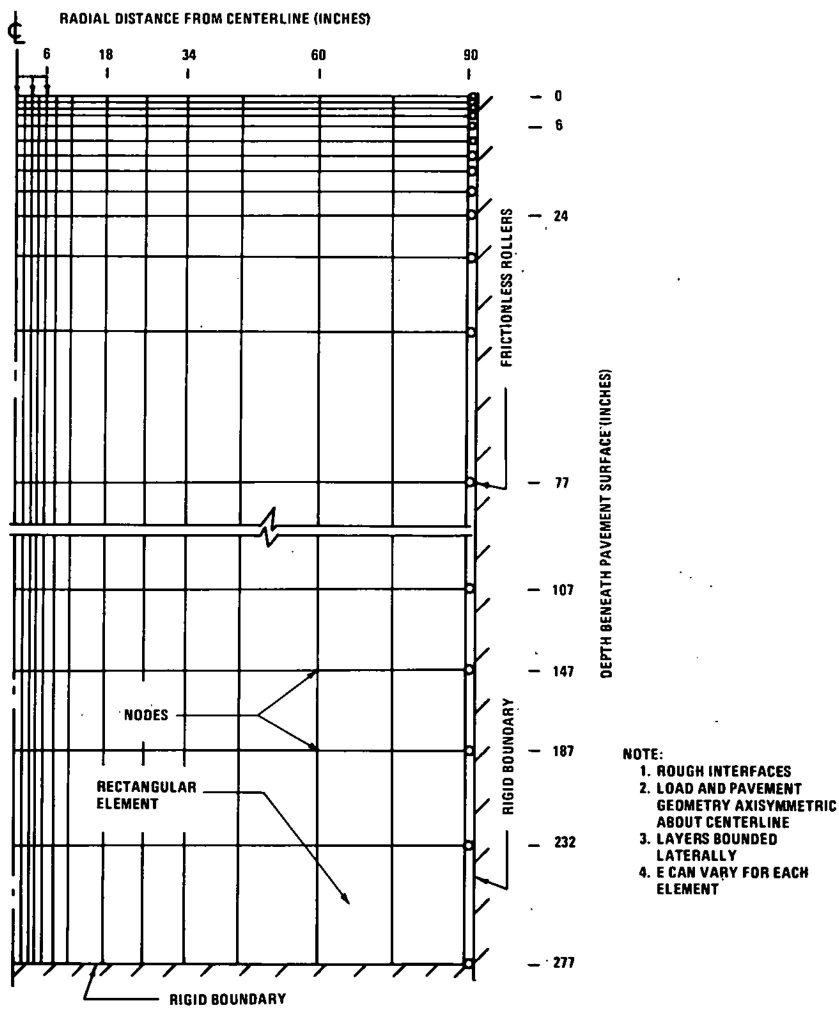


Figure 16. Finite-element idealization of pavement system.



theories considers the effects of vibrations. Neglecting vibrations is probably not a bad assumption for vehicle speeds lower than 60 mph on materials having cohesion. However, for cohesionless materials compacted to lower relative densities, neglecting vibratory effects may lead to densification that would cause rutting and changes in material properties.

Numerous laboratory tests have indicated that the dynamic modulus of paving materials varies with the confining pressure or deviator stress or both (1, 33, 55). Because of the variation in stress state that exists in each layer of the pavement system, the dynamic modulus actually changes with both depth and lateral position in each layer. Therefore, uncertainties arise in trying to determine what values of dynamic modulus to use in a linear-elastic layered analysis. Furthermore, elastic layered theory cannot consider variations in the modulus with lateral position. Those limitations for the most part can be overcome by the use of nonlinear finite-element theory (5, 17, 19, 33). In that method, the pavement response is initially calculated by using assumed moduli for each layer. The calculated stresses are then used to estimate a new stress-dependent modulus from experimentally measured material properties. Additional stress states are then calculated, and the process is repeated by either an iterative or an incremental procedure. In both cases, the modulus is matched with the stress state in each element. Those approaches, however, require considerably more computer time than does a single elastic layered solution.

An excellent alternative approach, which is a practical trade-off, is the use of a nonlinear, iterative elastic layered solution (1, 33, 38). That iterative procedure is analogous to the one used for finite-element theory. In this approach, the base and subgrade can be subdivided into several fictitious layers for better accuracy. The technique uses in each layer a modulus that is dependent on the average stress state that exists in the vicinity beneath the wheel loadings. That simplified nonlinear approach makes possible the analysis of a large number of pavement sections at a computer cost of roughly $10 for each structural section.

The bottom of each layer of a pavement structure is subjected to radial and tangential tensile stresses and strains. In nonstabilized granular materials, the application of such a stress state can result in an appreciable influence on the modulus of the material. A description of a slip model that can be used to handle that condition has been developed by the use of anisotropic finite-element theory (5). The behavior of granular materials under those stress conditions, however, certainly needs further study.

Prediction of Pavement Response

Classical Elastic Theory

One of the first attempts to verify layered theory was by Coffman et al. (12); they compared the deflections measured at the AASHO Road Test with deflections calculated from results of laboratory tests. A 3-layered elastic system was solved with tables prepared by Peattie and Jones (34, 54). Base, subbase, and subgrade materials were characterized by creep tests, and the results were transformed to a dynamic modulus. The dynamic modulus of the asphalt concrete surface was determined by a cyclic triaxial test. A typical comparison of calculated and measured deflections under normal traffic loads is shown in Figure 17 (12).

Brown and Pell (6) also verified elastic layered theory in carefully controlled laboratory experiments; they used a program developed by Jones (93). They found that elastic theory predicted vertical and maximum shear stress and maximum surface deflections satisfactorily. Klomp and Niesman (40) used the Shell Bistro program (57) to compare predicted strains in the asphalt-bound surface and base layers with values measured by wire resistance strain gauges. Flexural tests were used to determine the modulus of the asphalt-bound layers, and wave propagation techniques were used for the subgrade. In general, their study found that strain in the bound layers could be predicted with reasonably good accuracy. However, surface strain measurements traditionally have been more difficult to predict because of the effect of factors including tire profile, temperature variations, and tensile stresses in the asphalt concrete.

Seed et al. (65) also used classical elastic layered theory (35) to predict the response of layered systems consisting of asphalt concrete, granular base, and clay subgrade. Most materials were characterized by repeated-load triaxial tests although in some cases values of E for the asphalt concrete were determined by repeated flexural test (15). In each case, they were able to predict reasonably well the deflections occurring within a prototype pavement section. Kasianchuk (38) extended the approach used by Seed by developing an approximation-linear iterative technique for use with layered elastic theory. That technique has been employed by Barksdale (1), Hicks (33), and Hicks and Finn (34) to predict with varying accuracy the response of a wide variation of pavement sections. For example, Hicks (33), using classical elastic theory, was able to predict surface deflections and strains in the asphalt concrete, strains in the untreated base layer, and stresses in the subgrade with reasonably good accuracy. However, for the prototype pavement investigated, better comparisons were obtained with finite-element techniques. Hicks and Finn (34) used a similar procedure to predict the response of test sections at the full-scale San Diego test road. Typical comparisons between measured and predicted responses for one of the sections investigated (Fig. 18, 34) were reasonably good; however, other sections had ratios of predicted over measured values ranging from 0.4 to 1.4 for total deflection, 0.1 to 2.3 for surface strain, and 0.4 to 2.6 for strain within the asphalt layers.

Thrower et al. (84) have also presented evidence indicating that layered elastic theory can be used to predict stresses and strains in pavements subjected to moving wheel loads. Dynamic moduli of the asphalt concrete were measured by means of a dynamic flexural test. Wave propagation techniques were used to determine E of the base, and the subgrade E was determined by the Shell correlation procedure and the measured CBR value. They found good agreement between theory and measured response for stiff pavement sections. Serious discrepancies, however, occurred at high road temperatures partly because of difficulties in evaluating E of the asphalt concrete at high temperatures.

Finite-Element Approaches

Shifley (92) in 1967 and Duncan et al. (19) in 1968 used finite-element techniques to analyze pavement structures composed of an asphalt concrete surface, granular base, and clay subgrade. Shifley (92) conducted a series of rigid plate repeated-load tests on a full-scale test section to simulate slowly moving traffic. Transient deformations computed from the results of the repeated-load laboratory tests for multilayer pavement structures compared reasonably well with the measured test road deflections. The nonlinear behavior for both aggregate base and subgrade were approximately accounted for by an iterative finite-element procedure.

Duncan et al. (19) analyzed a pavement structure for both summer conditions (low asphalt stiffness) and winter conditions (high asphalt stiffness). Response of in-service pavements was calculated by an iterative finite-element program together with material properties determined from the repeated-load triaxial test. Predicted deflections were found to be in good agreement with those measured by the California traveling deflectometer, although some question exists concerning the convergence of the procedure.

A comprehensive study in which finite-element techniques were used has been performed by Dehlen (17) and Hicks (33). Both full-depth and conventional asphalt concrete structures were investigated. Dehlen made comparisons for the San Diego test road between measured stresses, strains, and deflection and those predicted by analytical and finite-element techniques. He found that, although the predicted and measured pavement responses were similar in shape, the quantitative agreement was only fair to good. Typical comparisons for surface deflections and strains within the asphalt concrete layer are shown in Figure 19. Hicks conducted similar analyses on both prototype (Fig. 20) and a conventional section on the San Diego test road and reached conclusions similar to those of Dehlen.

Figure 17. Comparison of measured and calculated surface deflections.

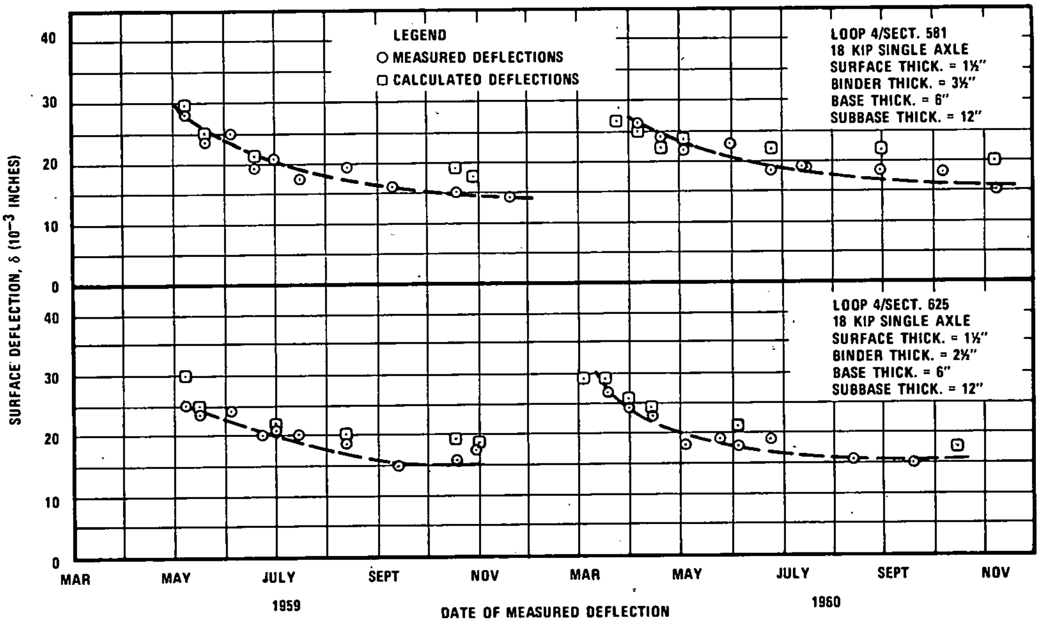
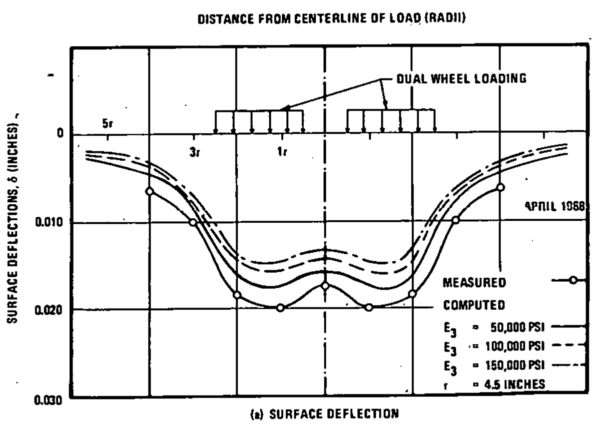


Figure 18. Comparison of measured and computed responses of test road section.



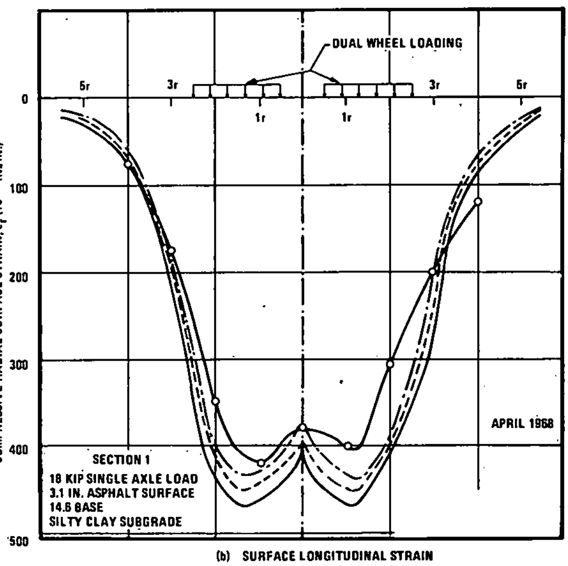
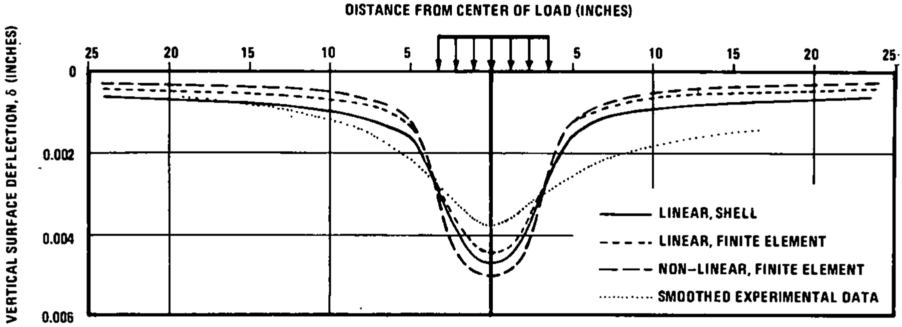
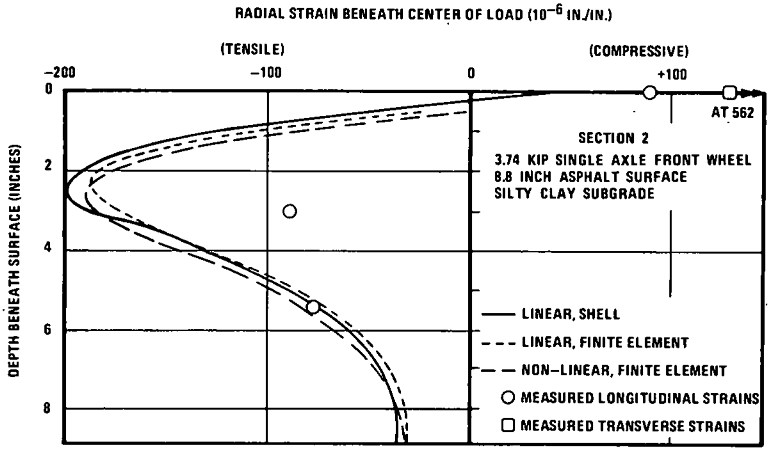


Figure 19. Comparison of measured and calculated surface deflections and radial strains.

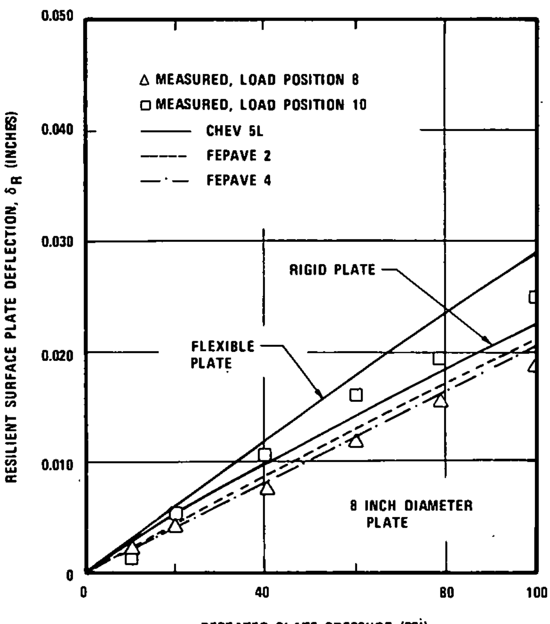


a. VERTICAL SURFACE DEFLECTIONS

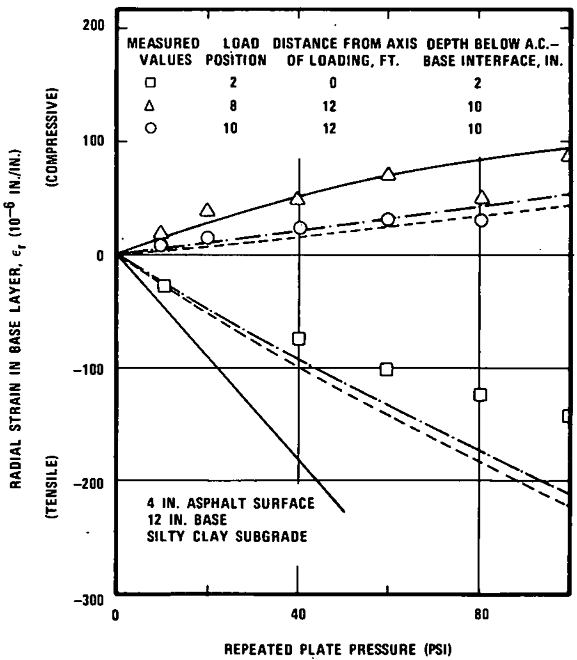


b. RADIAL STRAIN IN THE ASPHALT CONCRETE

Figure 20. Comparison of measured and computed responses at surface and base.



a. RESILIENT SURFACE DEFLECTION



b. RESILIENT RADIAL STRAIN IN THE BASE

Viscoelastic Approaches

Barksdale and Leonards (2) compared measured and computed responses by using a linear viscoelastic approach. They calculated both permanent and resilient deflections for 3-layer systems on the AASHO test road. The stress-strain relations were obtained from the results of repeated-load triaxial tests. The calculated deflections were in general close enough to suggest that viscoelastic theory can give reasonable estimates of resilient and permanent deformations. Drennon and Kenis (94) and Barksdale (5) have shown how a linear and nonlinear viscoelastic theory can be used to calculate layered system response.

Theory Selection

The studies just described show that layered theories give reasonably good predictions of deflections, stresses, and strains in layered pavement systems provided dynamic moduli are used that are compatible with the stress state in the layer. A number of elastic and finite-element computer programs that could be used in a mechanistic pavement design approach are now in operational form. Some of those programs and an indication of their availability and where they can be obtained are given in Table 3. A comparative study by Pichumani (89) of several of those programs is given in Table 4.

The selection of the most appropriate theory to use should depend on whether there is any benefit gained in using a more sophisticated approach. For example, a linear-elastic theory can usually be used with success for pavements having stabilized bases and a subgrade that exhibits a relatively linear response. In a general design procedure, however, a more flexible approach should probably be employed to account for the nonlinear behavior of some paving materials. A good example of such an approach is the modified Chevron 5-layer program, which provides for iteration to obtain moduli compatible with the stress state in the layer. Of course, use of a nonlinear finite-element approach would be more desirable and could be used if excessive computer time were not required in applying the program to a design method. Although linear-elastic theories require the least computer time and have been used by several organizations with apparently satisfactory results, considerable care must be taken in selecting appropriate material properties. For developing a general pavement design procedure, a linear-elastic program is recommended that has provisions for iteration.

SUMMARY

Suitable analytical procedures based on layered theory have been developed for predicting the response of flexible pavement structural sections. If applied, those methods have the potential for greatly improving the accuracy with which required structural sections can be selected for widely varying traffic, subgrade, and environmental conditions.

Material properties for use in solutions to layered theory should, where practical, be evaluated in a dynamic testing procedure. The dynamic loading can be readily accomplished by repeated loads to a cylindrical specimen inside a triaxial cell using a pneumatic loading system. The response of the pavement structure can then be predicted with reasonably good accuracy from the laboratory-determined material properties and either a linear-elastic computer program or an ad hoc non-linear-elastic program having iterative capability. Because the moduli of most pavement materials are stress dependent, the iterative approach is recommended.

Perhaps the most important problem in developing a mechanistic design approach is the inherent variability of material properties with depth and position along the project alignment. The ability to overcome that problem will depend to a great extent on how the properties are incorporated into the design procedure and how the procedure is related to actual field performance.

Predicting accurately the response of pavement structures can often be quite involved because of the nonlinear, inelastic, anisotropic nature of pavement materials, changes of material properties with time, uncertainties in conditions during construction,

Table 3. Operational computer programs and their availability.

Theory	Program	Availability	Source
Linear elastic (classical layered theory)	Shell BISTRO (axisymmetric)	Limited distribution (researchers only)	Shell Oil Company, One Shell Plaza, Houston, Texas 77002
	Chevron (axisymmetric)	No restrictions	R. J. Schmidt, Chevron Research Company, Richmond, California
Linear finite element	General axisymmetric (W1L67)	No restrictions	C. L. Monismith, University of California, Berkeley
	Prismatic space (2-dimensional)	No restrictions	University of California, Berkeley
	Extended 2-dimensional (AFPAV)	Under development	R. Pichumani, Kirtland Air Force Base, New Mexico
Nonlinear elastic (classical layered theory)	Chevron 5-layer elastic with iteration	No restrictions	University of California, Berkeley
Nonlinear finite element	FePave 1 (iterative)	No restrictions	University of California, Berkeley
	FePave 2 (incremental)	No restrictions	University of California, Berkeley
	Anisotropic incremental iterative	Available March 1973	R. D. Barksdale, Georgia Institute of Technology, Atlanta
Viscoelastic	Linear viscoelastic	No restrictions	F. Moavenzadeh, Massachusetts Institute of Technology, Cambridge

Table 4. Comparison of computer programs.

Description of Features	BISTRO	WIL67	VISAB3	AFPAV	Remarks
Theoretical basis	Burmister's layered system theory	Finite-element analysis of axisymmetric solids	Discrete-element analysis of pavement slabs on Winkler foundation	3-dimensional analysis of prismatic solids	AFPAV is called extended 2-dimensional finite-element code and cannot solve general 3-dimensional problems
Type of pavement analyzed	Rigid and flexible pavements	Rigid and flexible pavements	Rigid pavement only	Rigid and flexible pavements	Except for VISAB3, codes do not distinguish type of pavement
Number of pavement layers	N-layers (N = 10)	N-layers (N = 12)	2	N-layers (N = 15)	VISAB3 can treat only 2 layers, but the value of N can be changed by suitably modifying other codes
Number of applied loads	12 (can be increased)	1 (can be modified to analyze multiple loads)	Unlimited	Unlimited	BISTRO and WIL67 (when modified) use principle of superposition for multiple loads; VISAB3 and AFPAV consider multiple loads simultaneously
Contact pressure	Uniform over circular area	Uniform over circular area	Concentrated	Uniform over rectangular area	Nonuniform contact pressure can be treated approximately in AFPAV code
Joints in rigid pavement	Cannot consider	Cannot consider	Considers approximately	Can consider realistically	Joint analysis using AFPAV code is yet to be developed
Stress concentrations along joints	No	No	No	Yes	
Edge loads in rigid pavement	Cannot analyze	Cannot analyze	Can analyze	Can analyze	Edge-load analysis using AFPAV code is yet to be developed
Material properties	Linear elastic	Bilinear elastic	Linear elastic	Linear elastic	AFPAV code can be further developed to consider nonlinear material properties
Nonuniform layer thicknesses and properties	Cannot consider	Cannot consider	Cannot consider	Can consider	Changes referred to are in 1 vertical plane only and those should be constant in the longitudinal direction
Discontinuities (e.g., culverts) under pavement	Cannot analyze	Cannot analyze	Cannot analyze	Can analyze	Orientation of culvert should be normal to one horizontal axis of gear arrangement

uncertainties in environmental and subgrade conditions, and many other factors. As a result, the predicted behavior from any proposed pavement design method should be compared with the behavior observed in the field for as many pavement sections as possible. If the predicted behavior is not satisfactory, then the theory should be either revised or adjusted to give acceptable results. Provided the approach taken is theoretically sound, probably the most practical method will be to simply correlate the results with observed field performance (1, 82, 87) to develop design criteria such as limiting stresses and strains. Finally, design criteria developed by other investigators may not yield satisfactory results if either the method of material characterization or the method of calculating the response of the pavement structure is changed.

REFERENCES

1. Barksdale, R. D. Repeated Load Test Evaluation of Base Course Materials. Sch. of Civil Eng., Georgia Inst. of Tech., Dec. 1971.
2. Barksdale, R. D., and Leonards, G. A. Predicting Performance of Bituminous Surfaced Pavements. Proc., 2nd. Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 321-340.
3. Barksdale, R. D. Compressive Stress Pulse Times in Flexible Pavements for Use in Dynamic Testing. Highway Research Record 345, 1971, pp. 32-44.
4. Barksdale, R. D. Laboratory Evaluation of Rutting in Base Course Materials. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, 1972.
5. Barksdale, R. D. Analysis of Layered Systems. Sch. of Civil Eng., Georgia Inst. of Tech., July 1969.
6. Brown, S. F., and Pell, P. S. An Experimental Investigation of the Stresses, Strains and Displacements in a Layered Pavement Structure Subjected to Dynamic Loads. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967.
7. Brown, S. F., and Pell, P. S. A Fundamental Structural Design Procedure for Flexible Pavements. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, 1972.
8. Brown, S. F. Personal communication, Sept. 1972.
9. Burmister, D. M. The General Theory of Stresses and Displacements in Layered Systems. Jour. Appl. Phys., 1945.
10. Clark, D. C. On the Kinetic Behavior of Roads. HRB Proc., Vol. 40, 1961, pp. 179-204.
11. Coffman, B. S. Pavement Deflections From Laboratory Tests and Layer Theory. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 819-862.
12. Coffman, B. S., Craft, D. C., and Tamayo, J. A Comparison of Calculated and Measured Deflections for the AASHO Road Test. Proc., AAPT, Vol. 33, 1964, pp. 54-91.
13. Cost, T. L. Approximate Laplace Transform Inversions in Viscoelastic Stress Analysis. Jour. AIAA, Vol. 2, No. 12, Dec. 1964, pp. 2157-2166.
14. Deacon, J. A. Fatigue Life Prediction. Paper in this Special Report.
15. Deacon, J. A., and Monismith, C. L. Laboratory Flexural-Fatigue Testing of Asphalt Concrete With Emphasis on Compound Loading Tests. Highway Research Record 158, 1967, pp. 1-31.
16. Deacon, J. A. Materials Characterization—Experimental Behavior. HRB Spec. Rept. 126, Aug. 1970, pp. 150-179.
17. Dehlen, G. L. The Effect of Non-Linear Material Response on the Behavior of Pavements Subjected to Traffic Loads. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, graduate rept., 1969.
18. Dehlen, G. L., and Monismith, C. L. The Effect of Nonlinear Material Response on the Behavior of Pavements Under Traffic. Highway Research Record 310, 1970, pp. 1-16.
19. Duncan, J. M., Monismith, C. L., and Wilson, E. L. Finite Element Analyses of Pavements. Highway Research Record 228, 1968, pp. 18-33.
20. Dunlap, W. A. A Report on a Mathematical Model Describing the Deformation Characteristics of Granular Materials. Texas Transp. Inst., Texas A&M Univ., Tech. Rept. 1, 1963.

21. Durelli, A. J., Phillips, E. A., and Taso, C. H. Introduction to the Theoretical and Experimental Analysis of Stress and Strain. McGraw-Hill, New York, 1958, pp. 126-132.
22. Havens, J. H., Deen, R. C., and Southgate, H. F. Pavement Design Schema. Paper in this Special Report.
23. Finn, F. N., Nair, K., and Monismith, C. L. Application of Theory in the Design of Asphalt Pavements. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, 1972.
24. Fossberg, P. E. Load Deformation Characteristics of Three Layer Pavements Containing Cement Stabilized Base. Univ. of California, Berkeley, PhD thesis, Nov. 1970.
25. Gradowczyk, M. N., and Moavenzadeh, F. Characterization of Linear Viscoelastic Materials. Trans., Soc. of Rheology, Vol. 13, No. 2, 1969, pp. 173-191.
26. Gradowczyk, M. N., Moavenzadeh, F., and Soussou, J. F. Characterization of Linear Viscoelastic Materials Tested in Creep and Relaxation. Jour. Appl. Phys., Vol. 40, No. 4, March 1969, pp. 1783-1788.
27. Gregg, J. S., Dehlen, G. L., and Rigden, P. J. On the Properties, Behavior, and Design of Bituminous Stabilized Sand Bases. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 863-882.
28. Hadley, W. O., Hudson, W. R., Kennedy, T. W., and Anderson, V. L. A Statistical Experiment to Evaluate Tensile Properties of Asphalt-Treated Materials. Proc., AAPT, Vol. 38, 1969, pp. 224-241.
29. Hardin, B. O., and Drnevich, V. P. Shear Modulus and Damping in Soils—II: Design Equations and Curves. Coll. of Eng., Univ. of Kentucky, Tech. Rept. UKY27-70-CE3, July 1970.
30. Hardin, B. O., and Drnevich, V. P. Shear Modulus and Damping in Soils—I: Measurement and Parameter Effects. Coll. of Eng., Univ. of Kentucky, Tech. Rept. UKY27-70-CE2, July 1970.
31. Heukelom, W., and Klomp, A. J. G. Road Design and Dynamic Loading. Proc., AAPT, Vol. 33, 1964, pp. 92-125.
32. Heukelom, W., and Klomp, A. J. G. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, 1962, pp. 667-679.
33. Hicks, R. G. Factors Influencing the Resilient Properties of Granular Materials. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, Dissertation series, May 1970.
34. Hicks, R. G., and Finn, F. N. Analysis of Results From the Dynamic Measurements Program on the San Diego Test Road. Proc., AAPT, Vol. 39, 1970.
35. Jones, A. Tables of Stresses in Three-Layer Elastic Systems. HRB Bull. 342, 1962, pp. 176-214.
36. Kallas, B. F., and Riley, J. C. Mechanical Properties of Asphaltic Pavement Materials. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 931-952.
37. Kallas, B. F. Dynamic Modulus of Asphalt Concrete in Tension and Tension-Compression. Proc., AAPT, Vol. 39, 1970.
38. Kasianchuk, D. A. Fatigue Considerations in the Design of Asphalt Concrete Pavements. Univ. of California, Berkeley, PhD dissertation, 1968.
39. Kenney, J. T. Steady-State Vibrations of Beam on Elastic Foundation for Moving Load. Jour. Appl. Mech., Dec. 1954, p. 359.
40. Klomp, A. J. G., and Niesman, T. W. Observed and Calculated Strains at Various Depths in Asphalt Pavements. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967.
41. Ko, H. Y., and Scott, R. F. Deformation of Sand in Shear. Jour. Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM3, May 1967, pp. 137-156.
42. Larew, H. G., and Leonards, G. A. A Strength Criterion for Repeated Loads. HRB Proc., Vol. 41, 1962, pp. 529-556.
43. McCullough, B. F., Van Til, C. J., Vallerga, B. A., and Hicks, R. G. Evaluation of AASHO Interim Guide for Design of Pavement Structures. NCHRP Proj. 1-11, Final Rept., Dec. 1968.

44. Mitchell, J. K., and Chen, C. K. Soil-Cement Properties Determined by Repeated Loading in Relation to Bases for Flexible Pavements. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 427-451.
45. Elliott, J. F., and Moavenzadeh, F. Analysis of Stresses and Displacements in Three-Layer Viscoelastic Systems. Highway Research Record 345, 1971, pp. 45-57.
46. Monismith, C. L. Pavement Design: The Fatigue Subsystem. Paper in this Special Report.
47. Monismith, C. L., and Deacon, J. A. Fatigue of Asphalt Paving Mixtures. Transp. Eng. Jour., ASCE, No. TE2, May 1969, pp. 317-346.
48. Monismith, C. L., and McLean, D. B. Structural Design Consideration. Symp. on Tech. of Thick Lift Constr., AAPT, Feb. 1972.
49. Monismith, C. L., and Secor, K. E. Viscoelastic Behavior of Asphalt Concrete Pavements. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, 1962, pp. 476-498.
50. Nair, K. Solutions and Solution Techniques for Boundary Value Problems. HRB Spec. Rept. 126, 1970, pp. 103-113.
51. Nair, K., Smith, W. S., and Chang, C. Y. Characterization of Asphalt Concrete and Cement-Treated Granular Base Courses. Materials Research and Development, Inc., Oakland, Calif., Feb. 1972.
52. Nair, K., and Chang, C. Y. Materials Characterization. Materials Research and Development, Inc., Oakland, Calif., 1970.
53. Papazian, H. S. The Response of Linear Viscoelastic Materials in the Frequency Domain With Emphasis on Asphaltic Concrete. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, 1962, pp. 454-463.
54. Peattie, K. R. Stress and Strain Factors for Three-Layer Elastic Systems. HRB Bull. 342, 1962, pp. 215-253.
55. Pell, P. S., and Brown, S. F. The Characteristics of Materials for the Design of Flexible Pavement Structures. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, 1972.
56. Pell, P. S. Characterization of Fatigue Behavior. Paper in this Special Report.
57. Peutz, M. G. F., van Kempen, H. P. M., and Jones, A. Layered Systems Under Normal Surface Loads. Highway Research Record 228, 1968, pp. 34-45.
58. Sayegh, G. Viscoelastic Properties of Bituminous Mixtures. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967, pp. 743-755.
59. Schapery, R. A. A Simple Collocation Method for Fitting Viscoelastic Models to Experimental Data. Guggenheim Aeronaut. Lab., Calif. Inst. of Tech., SM-61-23A.
60. Schapery, R. A. Approximate Methods of Transform Inversion for Viscoelastic Stress Analysis. Proc., 4th U.S. Natl. Congr. of Appl. Mech., ASME, Vol. 2, 1962, pp. 1075-1085.
61. Schiffman, R. L. General Analysis of Stresses and Displacements in Layered Elastic Systems. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, 1962.
62. Schiffman, R. L. The Use of Viscoelastic Stress-Strain Laws in Soil Testing. ASTM, Spec. Tech. Publ. 254, 1960, pp. 131-155.
63. Schmidt, R. J. A Practical Method for Measuring the Resilient Modulus of Asphalt Treated Mixes. Highway Research Record 404, 1972, pp. 22-31.
64. Seed, H. B., and Chan, C. K. Effect of Stress History and Frequency of Stress Applications on Deformations of Clay Subgrades Under Repeated Loading. HRB Proc., Vol. 37, 1958, pp. 77-87.
65. Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K. Factors Influencing the Resilient Deformations of Untreated Aggregate Base in Two-Layer Pavements Subjected to Repeated Loadings. Highway Research Record 190, 1967, pp. 19-57.
66. Shook, J. F., and Kallas, B. F. Factors Influencing Dynamic Modulus of Asphalt Concrete. Proc., AAPT, Vol. 38, 1969, pp. 131-155.
67. Stephenson, R. W., and Monke, P. G. Ultrasonic Moduli of Asphalt Concrete. Highway Research Record 404, 1972, pp. 8-21.
68. Terrel, R. L. Factors Influencing the Resilient Characteristics of Asphalt Treated Aggregates. Univ. of California, Berkeley, PhD dissertation, Aug. 1967.
69. Terrel, R. L. Examples of Approach and Field Application: Research Applications. Paper in this Special Report.

70. Terrel, R. L., and Awad, I. S. Laboratory Considerations. Symp. on Tech. of Thick Lift Constr., AAPT, Feb. 1972.
71. Thompson, M. R. The Split-Tensile Strength of Lime-Stabilized Soils. Highway Research Record 92, 1965, pp. 11-23.
72. Van der Poel, C. A General System Describing the Viscoelastic Properties of Bitumens and Its Relation to Routine Test Data. Jour. Appl. Chem., May 1954, pp. 221-236.
73. Van der Poel, C. Road Asphalt, in Building Materials, Their Elasticity and Inelasticity. Interscience Publishers, New York, 1954, pp. 361-443.
74. Van Draat, W. E. F., and Sommer, P. Ein Gerat zur Bestimmung der dynamischen Elastizitatsmoduln von Asphalt. Strasse und Autobahn, Vol. 35, 1965, pp. 206-211.
75. Verstraeten, J. Stresses and Displacements in Elastic Layered Systems. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967.
76. Warren, H., and Dieckman, W. L. Numerical Computation of Stresses and Strains in a Multiple Layer Asphalt Pavement System. Chevron Research Corp., Unpubl. intern. rept., 1963.
77. Westergaard, H. M. Stresses in Concrete Pavements Computed by Theoretical Analysis. Public Roads, Vol. 7, No. 2, 1926, pp. 25-35.
78. Williams, M. L., Blatz, P. J., and Schapery, R. A. Fundamental Studies Relating to Systems Analysis of Solid Propellants. Guggenheim Aeronaut. Lab., Calif. Inst. of Tech., SM-61-5, Feb. 1961.
79. Williams, M. L. Structural Analysis of Viscoelastic Materials. Jour. AIAA, Vol. 2, No. 5, May 1964, pp. 785-808.
80. Williams, M. L., Blatz, P. J., and Schapery, R. A. Fundamental Studies Relating to Systems Analysis of Solid Propellants. Guggenheim Aeronaut. Lab., Calif. Inst. of Tech., Final Rept. GALCIT 101, Feb. 1961.
81. Wilson, E. L. A Digital Computer Program for the Finite Element Analysis of Solids With Non-Linear Material Properties. Dept. of Civil Eng., Univ. of California, Berkeley, Unpubl., 1965.
82. Witczak, M. Fatigue Subsystem Solution for Asphalt Concrete Airfield Pavements. Paper in this Special Report.
83. Bishop, A. W., and Henkel, D. J. The Measurement of Soil Properties in the Triaxial Test. Edward Arnold, Ltd., London, 1962.
84. Thrower, E. N., Lister, N. W., and Potter, J. F. Experimental and Theoretical Studies of Pavement Behavior Under Vehicular Loading in Relation to Elastic Theory. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, 1972.
85. Wang, M. C. Stress and Deflections in Cement Stabilized Soil Pavements. Univ. of California, Berkeley, PhD dissertation, 1968.
86. Kirnan, R. W., and Glynn, T. E. Investigation of Soils Subjected to Dynamic Loading. Eng. Sch., Univ. of Dublin, Dec. 1968.
87. 1963 Design Charts for Flexible Pavements. Shell Oil Co., New York, 1963.
88. Richards, B. G. Moisture Flow and Equilibrium in Unstabilized Soils for Shallow Foundations. In Permeability and Capillarity of Soils, ASTM, STP 417, 1967.
89. Pichumani, R. Theoretical Analysis of Airfield Pavement Structures. Air Force Weapons Lab., Kirtland Air Force Base, Tech. Rept. AFWL-TR-71-26, July 1971.
90. Westermann, R. A. Viscoelastic and Thermelastic Analysis of Layered Systems. Univ. of California, Berkeley, PhD dissertation.
91. Zienkiewicz, O. C., and Cheung, Y. K. The Finite Element Method in Structural and Continuum Mechanics. McGraw-Hill, London, 1967.
92. Shifley, L. H., Jr. The Influence of Subgrade Characteristics on the Transient Deflections of Asphalt Concrete Pavements. Univ. of California, Berkeley, PhD dissertation, 1967.
93. Jones, A. The Calculation of Stresses, Strains and Displacements in Layered Systems Having Constant and Variable Elastic Parameters. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, 1967.
94. Drennon, C. B., and Kenis, W. J. Response of a Flexible Pavement to Repetitive and Static Loads. Highway Research Record 337, 1970, pp. 40-54.

This paper reviews the state of knowledge resulting from laboratory research into the phenomenological characterization of fatigue behavior. In particular, it considers the influence of various factors that affect the fatigue performance of asphalt mixes. The definition of laboratory service life and the values obtained depend on the method of testing; controlled stress loading appears to be applicable for materials used in thick asphalt construction, and controlled strain loading is more appropriate for thin layers. Fatigue performance can best be characterized by a strain ϵ and life N_s relation of the form $N_s = C(1/\epsilon)^m$. The factors C and m depend on the composition and properties of the mix and are also affected by the testing method. Under controlled stress conditions, mixes having maximum stiffness will give longer lives; and, therefore, the choice of mix composition should be such that under compaction maximum tensile stiffness associated with minimum voids is obtained. Under controlled strain condition, longer lives are likely to be obtained from more flexible, less stiff materials. Ideally, general strain-life relations can be established for various mixes by laboratory testing and used for design purposes. Alternatively, use may be made of empirical relations relating fatigue performance to mix properties.

Characterization of Fatigue Behavior

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The term "fatigue" implies a mode of distress in an asphalt concrete pavement resulting from the repeated application of traffic-induced stresses. In particular, this symposium was concerned with fatigue fracture, and the purpose of this paper is to consider the influence of various factors that affect the fatigue performance of asphalt mixes. A considerable amount of laboratory research has been carried out and documented (1, 2, 3, 4, 5, 6) on the phenomenological characterization of fatigue and enables certain overall conclusions to be drawn. As well as providing useful information for optimum mix design against fatigue cracking and a means of comparing the performance of different mixes, the current state of the art can be used to predict that cracking will occur after the application of a particular traffic volume.

However, a considerable amount of crack propagation will have to occur over a wide area before any serious deterioration of the pavement structure results from the possible penetration of water or from increase in stresses in the underlying layers or from both. Pavement performance under traffic depends not solely on the characteristics of the materials in the individual layers but rather on the interaction of the various layers, and that aspect of the problem is discussed in other papers (30, 31).

Besides the phenomenological approach to the problem of fatigue of asphaltic concrete pavements with which this paper is concerned, a more recent development is the mechanistic approach based on fracture mechanics (7), and that is discussed in another paper (33). Although the primary object of this present paper is to emphasize the existing state of knowledge that can be applied, certain areas requiring further research in order to improve the accuracy of fatigue life predictions will be mentioned.

DEFINITIONS

Fatigue has been defined (2) as "the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material." However, in practice and in the laboratory, fatigue failure is often loosely considered to be the point at which the material or specimen is unable to continue to perform in a satisfactory manner. The failure or the end point of a fatigue test has been defined by investigators in many ways. It may be the point corresponding to complete fracture of the test specimen, the point at which a crack is first observed or detected, or the point at which the stiffness or some other property of the specimen has been reduced by a specific amount from its initial value. The choice is often arbitrary depending on the method of testing being used, and hence an intelligent interpretation of fatigue results requires that the point of failure be explicitly defined.

Service life N_s is the accumulated number of load applications necessary to cause failure in the test specimen. In general, the service life as defined here has often been called the fatigue life, but it is a function of the manner in which failure is defined. Fracture life N_f is the accumulated number of load applications necessary to completely fracture a specimen. When the failure point is complete fracture, then the service and fracture lives are identical.

The fatigue behavior of a specimen subjected to repeated loading depends primarily on the load, environmental, and specimen variables. Load condition refers to the particular set of values that the appropriate load and environmental variables assume for a particular load application. Simple loading occurs when the load condition remains unchanged throughout the fatigue test. Compound loading results from the repeated application of loads in which the load condition changes during the fatigue test. In practice, asphalt pavements are subjected to a form of compound loading with a succession of load pulses of varying sizes and durations and with varying time intervals between pulses depending on the details of the traffic. Changes, such as ambient temperature and moisture conditions, are also taking place in the environment. Few attempts have yet been made in the laboratory to study cumulative damage by compound loading, and most laboratory fatigue tests have been of the simple loading type with the load-time curve being sinusoidal, triangular, square, or some other regular wave form.

TYPE OF FATIGUE TEST

Generally, testing methods are either controlled stress mode when the loading is in the nature of an alternating stress of constant amplitude or controlled strain mode when the loading is in the form of an applied alternating strain or deflection of constant amplitude (Fig. 1). In controlled stress loading, if the stiffness of the specimen reduces during the test, then the strain will gradually increase; in controlled strain, the resulting stress on the specimen will fall with decreasing stiffness. In many cases, the service life of the specimens greatly depends on which of the 2 modes of testing is used.

Because the method of testing influences the results of fatigue tests, the question arises as to which method is preferable. Monismith and Deacon (2) have proposed a more quantitative basis for differentiation between the 2 modes of loading by introducing the mode factor, a parameter defined as

$$\text{Mode factor} = \frac{|A| - |B|}{|A| + |B|} \qquad (1)$$

where $|A|$ and $|B|$ are the percentage changes in stress and strain respectively for an arbitrary but fixed percentage reduction in stiffness. The mode factor has a value of -1 for controlled stress loading and +1 for controlled strain conditions. For intermediate modes, where both stress and strain are changing during the test, the mode factor lies between the limits of -1 and +1.

The applicability of types of fatigue tests to actual road conditions has been considered in various types of pavement construction analysis using layered elastic theory to investigate the effect of variations in asphalt stiffness on the stresses and strains occurring in the asphalt layer. Figure 2 (2) shows a typical example of a 3-layer structure where E_2 = 20,000 psi, E_3 = 6,000 psi, and $h_1 + h_2$ = 26 in. As the thickness and stiffness of the asphalt layer increase, the mode factor decreases and a controlled stress condition is approached. It is, therefore, suggested that controlled stress testing conditions are appropriate for thicker asphalt layers, say, 6 in. or more, and controlled strain tests are suitable for thin asphalt layers of 2 in. or less, for under those conditions the strain is little affected by the mixture stiffness.

For the intermediate thicknesses, some form of testing between those 2 extreme modes would strictly be appropriate; but for an engineering design approach, controlled stress tests would seem sensible because they give a conservative estimate of fatigue life.

The method of performing simple loading fatigue tests is to test specimens in controlled stress loading at different stress levels (or in controlled strain loading at different strain levels) and to determine the corresponding service life. A variety of testing equipment has been used; each type of apparatus and its associated specimen size and shape has certain advantages and disadvantages. To date, most results have been obtained from bending or flexure tests on rectangular specimens tested as simply supported beams (1, 5, 8, 9, 10), trapezoidal-shaped specimens tested as cantilevers (11, 12), or specimens having a circular cross section with varying diameter tested as rotating cantilevers (4, 6, 13). Cylindrical and rectangular specimens are increasingly being used both under direct uniaxial (tension-compression) loading (13, 14) and under triaxial states of stress (15). Plate specimens (16) and torsional specimens (17) have been used to obtain biaxial stress conditions, and the indirect tensile test has also recently been adapted for repetitive loading (18).

Most laboratory fatigue tests have been carried out under uniaxial stress conditions either in flexure or in direct loading, but in the pavement the material is subjected to complex, 3-dimensional stressing (13). In the case of bituminous-bound materials, it is not expected that the effect of confining stress and shear reversal will be as significant as with unbound materials; but nevertheless fatigue tests under more realistic stress conditions need to be carried out to confirm that expectation (12).

PRESENTATION OF FATIGUE TEST RESULTS

There is always considerable scatter of results in any fatigue testing of nominally identical specimens, and that is particularly so in the case of asphaltic mixtures because of the inherent inhomogeneity of the material and the unavoidable variation in specimen preparation. That means that fatigue life must be considered in a statistical manner and strictly as only a distribution of individual values. It is usually assumed that there is a logarithmic normal distribution of fatigue lives at a particular loading condition, and a histogram (Fig. 3) constructed from the results of tests on 100 nominally identical specimens tested under the same loading conditions supports that assumption (4).

Because of the normal scatter, it is necessary to test several specimens at each stress or strain level, and the results are usually plotted on log-log scales as stress or strain versus cycles of load to failure. Figure 4 shows some typical results of controlled stress testing. Individual lives are plotted at +10 C, and it can be seen that a straight line passes through the mean of the logarithm of the lives at each stress level. That type of result has generally been found to be true, and an equation representing that relation can be written as

$$\log N_s = \log K - n \log \sigma \qquad (2)$$

where

N_s = mean service life obtained at particular loading conditions,
σ = amplitude of applied tensile stress, and
K and n = coefficients that can be determined by linear regression analysis techniques.

Figure 1. Types of fatigue tests.

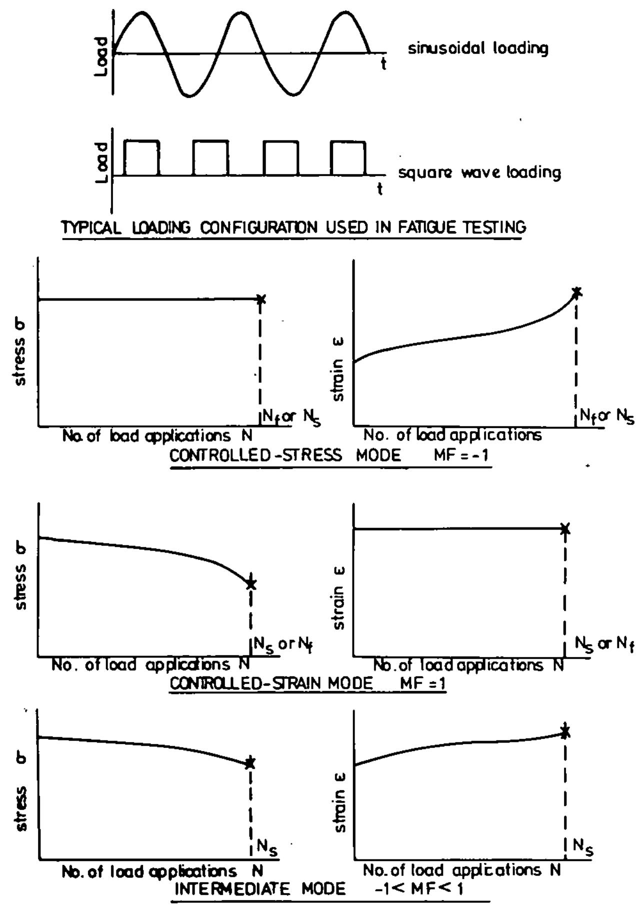
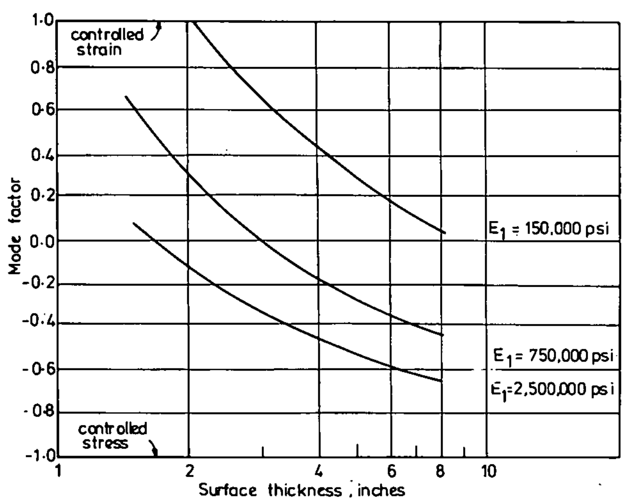


Figure 2. Variation of mode factor with surface thickness.



Equation 2 may be expressed as

$$N_s = K\left(\frac{1}{\sigma}\right)^n \qquad (3)$$

The exponent n defines the slope of the fatigue line; lower values of n denote a steeper line.

A similar type of relation but one in terms of applied tensile strain ϵ is obtained from controlled strain tests.

There is no evidence of an endurance limit up to lives of 10^7 applications of load, but the slope of the fatigue line is such that a small change in stress level can result in a considerable change in life.

EFFECT OF STIFFNESS AND CRITERION OF FAILURE

Possibly the greatest difficulty in interpreting fatigue test results arises from the fact that they are influenced by the method of testing. That is well illustrated by the effect of stiffness on the service life of identical specimens tested in both controlled stress and controlled strain.

If specimens are tested in controlled stress, such as in a rotating bending type of machine producing a sinusoidally varying bending stress of constant amplitude, then 4 different stiffness results, such as those shown in Figure 5a, are obtained. At a particular stiffness $\bar{S}$, the mean fatigue lives can be represented by a straight line on a log-log plot of stress σ against number of cycles of load N_s to cause failure. Different stiffnesses are represented by parallel lines showing that, with this type of testing, the fatigue life is highly dependent on stiffness; the stiffer the mix is, the longer the life is.

The stiffness, defined as the ratio of stress amplitude to strain amplitude, is dependent on the temperature and speed of loading. If the results of the fatigue tests under controlled stress are replotted in terms of strain ϵ, as shown in Figure 5b, it has been found that for a wide temperature range all the results from different stiffnesses coincide, indicating that strain is a major criterion of failure and that the effects of temperature and speed of loading can be accounted for by their effect on stiffness. There is some evidence that at higher temperatures, above about 25 C, longer lives are obtained that cannot be explained in this manner.

If identical specimens are tested in a controlled strain machine, which applies an alternating strain of constant amplitude, results such as those shown in Figure 5c are obtained. Although the lines at high stiffnesses, S_1 and S_2, say, coincide, those at lower stiffnesses show an effect of stiffness that is the reverse of that found from controlled stress tests.

The reason is that the mode of failure is different in the 2 types of test. In the controlled stress test, the formation of a crack results in an increase in actual stress at the tip of the crack due to the stress concentration effect, and that leads to rapid propagation and complete fracture of the specimen and termination of the test. In the constant strain test, on the other hand, cracking results in a decrease in stress and hence a slow rate of propagation. At low stiffnesses and, hence, low stresses, the measured fatigue life includes a considerable length of time necessary to propagate a crack or cracks sufficiently to reach an arbitrary state when the specimen is considered to have failed (service life).

If measurements of stiffness are taken during a controlled strain test, the stiffness reduces with increasing number of load applications at low stiffnesses, i.e., high temperatures; and that, no doubt, is partially due to formation of small cracks. At high stiffnesses, coincident with lines for S_1 and S_2 (Fig. 5c), there is negligible fall in stiffness during a fatigue test.

In some types of controlled stress tests, there is little increase in deflection and hence strain during the test even at low stiffnesses, but other types of controlled stress tests show a decrease in stiffness. Therefore, when the results are plotted in terms of strain, it is usual to take the value of stiffness of the specimen at the start of the test and quote the initial strain.

Figure 3. Results of 100 fatigue tests under 1 loading condition.

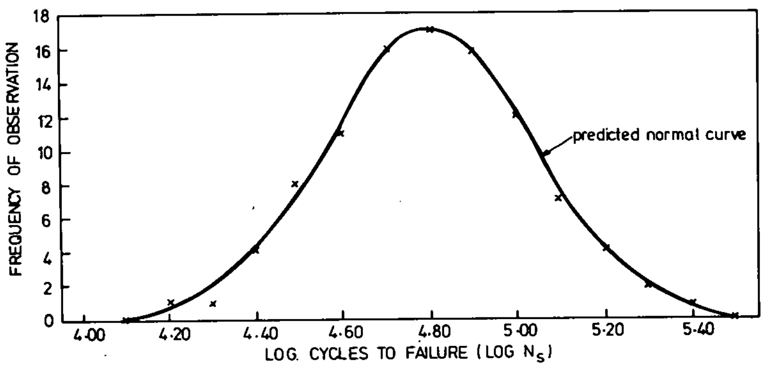


Figure 4. Results of fatigue tests under controlled stress at various temperatures.

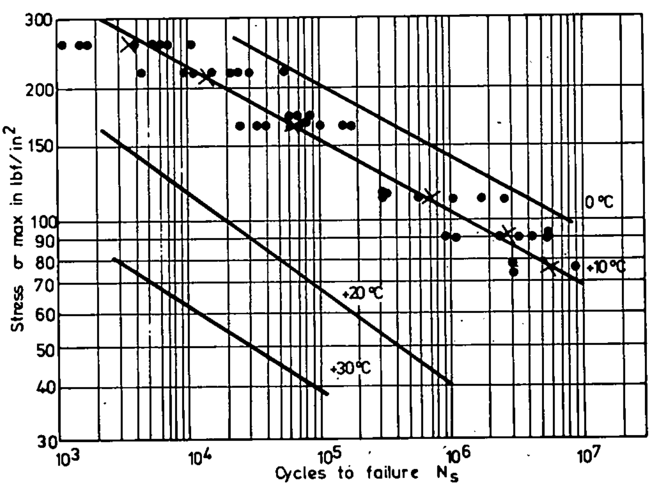
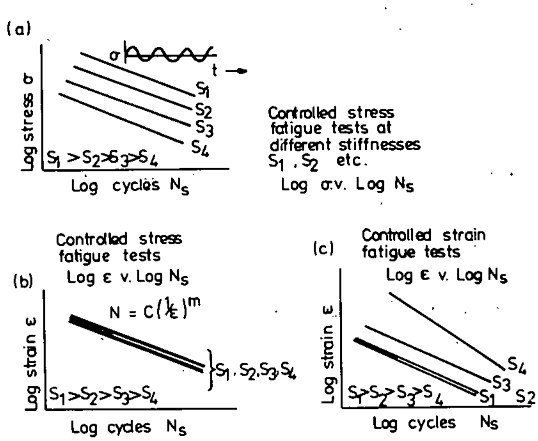


Figure 5. Effect of stiffness on fatigue life under different modes of loading.



Thus, it can be seen that the measurement of fatigue life is complicated by changes in stiffness that take place during a test and that are due to either the particular strain pattern or the propagation of small cracks or both. If the service life contains a lot of crack propagation time, then the simple criterion shown in Figure 5b, which applies essentially to crack initiation, no longer holds.

That criterion of fatigue crack initiation is one of applied tensile strain, and a general relation defining the fatigue life is as follows:

$$N_s = C\left(\frac{1}{\epsilon}\right)^m \tag{4}$$

where

N_s = number of applications of load to initiate a fatigue crack,
ϵ = amplitude of applied tensile strain, and
C and m = factors depending on the composition and properties of the mix.

For many dense mixes, the slope factor m has a value of approximately 5 or 6; but certain mixes, particularly those containing softer grades of bitumen, give steeper lines even under controlled stress testing that includes very little crack propagation time. Some typical results for different mixes are shown in Figure 6; the details of the composition of the mixes are given in Table 1.

The slope of the fatigue line appears to depend on the stiffness characteristics of the mix and the nature of the binder; mixes having high stiffnesses and linear behavior give a flatter line. That type of behavior is characteristic of dense surface-course mixes having a relatively high binder content of a harder bitumen. The leaner base-course mixes made with softer grades of binder show considerable nonlinearity, particularly at higher stress levels, and those mixes have a steeper fatigue line.

Although the logarithmic strain-life relation is usually shown as a straight line, it is probably curvilinear, particularly at high strains where nonlinearity is apparent.

If the method or conditions of testing are such that considerable crack propagation takes place during the test, then the line representing the service lives of specimens will be steeper as shown in Figure 7 because the rate of crack propagation depends on the stress level. That is likely to occur at higher temperatures (lower stiffnesses), particularly under controlled strain testing. However, a relation similar to Eq. 4 will still define the fatigue characteristics of the mix, but the values of factors C and m will be different.

EFFECT OF MIX VARIABLES

From the foregoing it will be realized that stiffness plays a predominant role in determining the fatigue behavior of bituminous mixes. It appears that maximum principal strain is a good criterion of crack initiation; and, therefore, in controlled stress tests, the stiffness will determine the strain level and hence the fatigue life. In controlled strain tests, which include crack propagation time in the measured life, stiffness again is important for it controls the stress level that determines the rate of crack propagation.

In general, increased stiffness results in longer lives at a given stress level in controlled stress testing and shorter lives in controlled strain testing at a given strain level.

It, therefore, follows that any mix variables that affect the stiffness are also going to affect the fatigue life of asphalt mixes. Those variables are aggregate type and grading, including filler, binder type, hardness (viscosity) and content, degree of mix compaction, and resulting air void content. The 2 factors that appear to be of primary importance are binder content and voids content.

Increasing voids reduces the fatigue life markedly (Fig. 8). The effect of increasing voids is twofold: reduced stiffness and increased stress concentrations due to the presence of voids in the material. Therefore, the detrimental effect of voids is likely to be more apparent in controlled stress testing or controlled strain testing at low temperatures. If increasing the bitumen content reduces the voids, then the fatigue life will be

Figure 6. Strain-life fatigue results for various mixes from controlled stress testing.

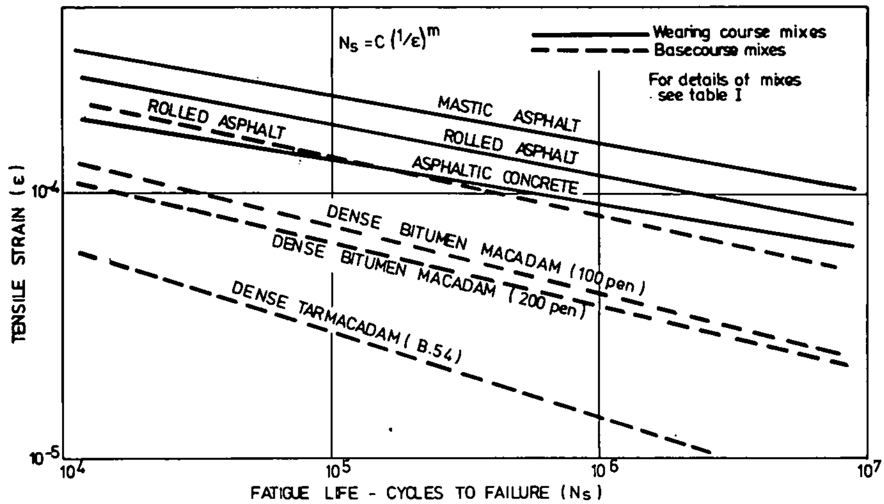


Table 1. Typical mixes tested in controlled stress.

Description of Mix	Coarse Aggregate[a] (percent by wt)	Fine Aggregate (percent by wt)	Filler (percent by wt)	Binder Percent by Wt	Binder Penetration	Mean Voids (percent)	C	Slope Factor
Mastic asphalt wearing course	42	23[b]	20[b]	15	70/30 TLA/20	0	1.13×10^{-15}	5.5
Rolled asphalt wearing course, BS 594, gap graded	30	52.2[c]	8.9[b]	7.9	45	2.9	8.8×10^{-15}	5.1
Asphaltic concrete wearing course, continuously graded	42	46.8[a]	4.7[a]	6.5	70	3.6	2.2×10^{-19}	6.1
Rolled asphalt base course BS 594, gap graded	65	29.3[c]	–	5.7	45	4.0	6.7×10^{-12}	4.2
Dense bitumen macadam base course, MOT spec., continuously graded	62	28.6[a]	4.7[a]	4.7	100	6.8	1.9×10^{-11}	3.8
Dense bitumen macadam base course, MOT spec., continuously graded	62.3	28.7	4.7[a]	4.3	200	6.9	1.8×10^{-12}	4.0
Dense tar macadam base course, MOT spec., continuously graded	61.7	28.4	4.7[a]	5.2	B 54	7.5	2.7×10^{-9}	3.0

[a]Crushed rock. [b]Limestone. [c]Sand.

Figure 7. Effect of crack propagation on the slope of the strain-life fatigue relation.

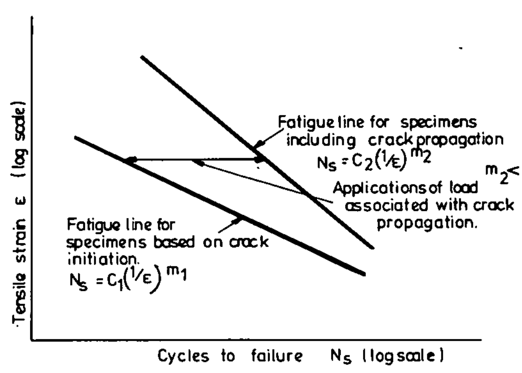
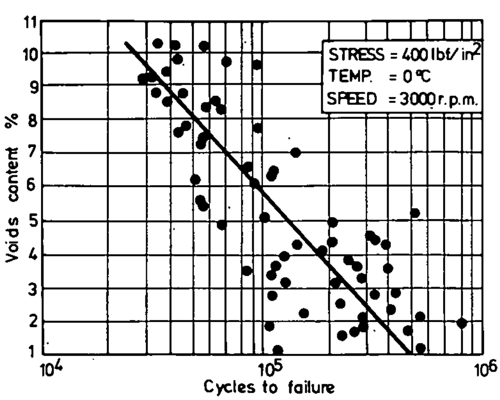


Figure 8. Effect of void content on fatigue life in controlled stress testing.



increased; but if the mix already has negligible voids, then further binder will reduce the stiffness and result in increased strain and hence reduced lives under controlled stress testing (Fig. 9).

The general effect on the strain-life relation of altering the binder and filler content of a particular mix is shown schematically in Figure 10. For a lean mix, increasing the binder and filler will result in a stiffer material and hence smaller strains and longer life. However, if too much binder is added, the stiffness is reduced and hence an optimum fatigue life exists.

Although other mix variables such as aggregate type and grading do affect the fatigue performance of asphaltic mixes, they can be largely accounted for by their effect on the 3 main factors: stiffness, binder content, and voids content. Figure 11 shows the results of 13 similar gap-graded, rolled-asphalt, base-course mixes made with different aggregates and binders as given in Table 2. The plotted points give the mean lives obtained at different stress levels and represent more than 400 individual fatigue tests. The general conclusion is that for mixes with similar binder contents aggregate type has little effect on the strain-life relation.

Figure 12 shows the strain-life lines for some continuously graded asphaltic concrete mixes made with different aggregates and binders. The important effect of binder content and void content is evident. Similar overall conclusions on the effect of mix factors on fatigue life have been presented by Epps and Monismith (5). There is some evidence that asphalt type does affect the strain-life relation; harder grades give slightly improved performance under controlled stress conditions. The importance of asphalt viscosity on controlled strain fatigue results has been shown by Santucci and Schmidt (19).

In conclusion, it may be stated that for good fatigue performance for thick asphalt construction a mix of maximum stiffness should be the objective and the quantities of filler and binder should be such that a condition of maximum tensile stiffness associated with minimum voids is produced.

EFFECT OF REST PERIODS

The fatigue characteristics discussed above have been obtained from tests carried out under simple loading conditions that mainly apply continuous cycles of loading of particular magnitudes. In practice, the material is subjected to a succession of load pulses of varying sizes and at varying time intervals between pulses depending on the details of the traffic. The question, therefore, arises as to the possible beneficial effect of periods of rest during a fatigue test.

Some workers (4, 17) have reported crack-initiation life did not increase in asphaltic mixes as a result of periods of rest that were at different temperatures and injected after varying portions of the expected life but did significantly increase in specimens made from bitumen alone under similar testing conditions. Bazin and Saunier (11), on the other hand, report that asphaltic concrete made with a very soft binder (200-penetration bitumen) had increased lives because of healing following rest periods under compressive stress.

More recent work reported by Raithby and Sterling (14, 20) and by Van Dijk et al. (12) show considerable beneficial effects of strain recovery if periods of rest are injected between each load pulse. Those findings mean that laboratory tests using continuous cycling load pulses may well underestimate the fatigue life to cause initiation of cracks in practice.

CUMULATIVE DAMAGE

In practice, asphalt pavements are subjected to a form of compound loading, and changes take place in the loading conditions during the life. In a recent review of some general cumulative damage theories, O'Neill (21) concludes that none of the hypotheses considered shows a clear general superiority to the rule of linear summation of cycle ratios. When satisfactory, constant-amplitude, simple load test data can be provided, the application of the linear rule, generally referred to as Miner's rule, is extremely simple.

Figure 9. Effect of binder content on fatigue life in controlled stress testing of continuously graded asphaltic mix using 200-penetration binder.

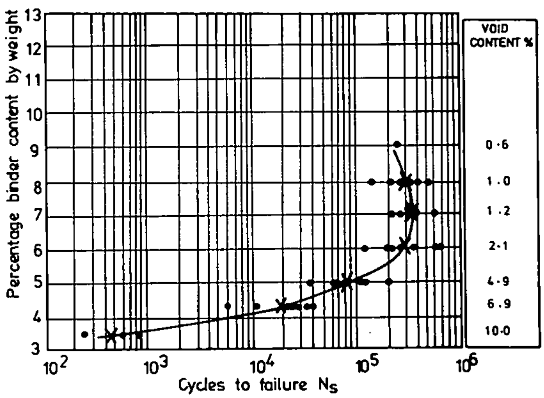


Figure 10. Effect of increasing binder and filler contents on fatigue life (strain criteria).

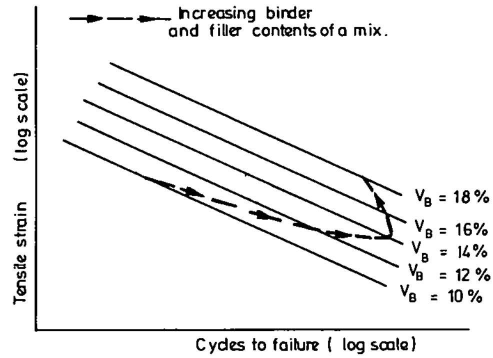


Figure 11. Effect of mix variables on fatigue performance of rolled asphalt.

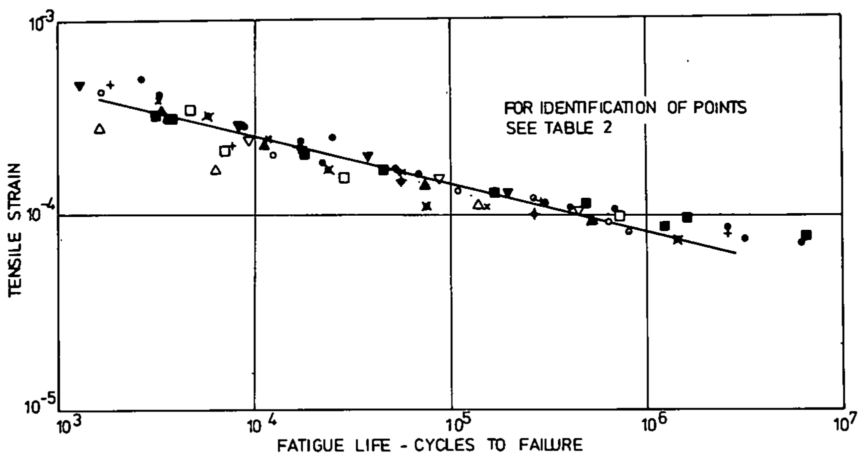


Table 2. Rolled asphalt base-course mixes.

Coarse Aggregate		Fine Aggregate		Filler	Binder		Mean	
Type	Percent by Wt	Type	Percent by Wt	(percent by wt)	Percent by Wt	Penetra-tion	Voids (percent)	Symbol on Figure 11
Crushed rock	60	Sand	34	0	6	45	5.0	●
Crushed rock	60	Crushed rock	34	0	6	45	5.6	○
Crushed rock	60·	Sand	34	0	6	35	4.2	✖
Crushed rock	65	Sand	29.3	0	5.7	45	4.0	✚
Crushed rock	60	Sand	34	0	6	100	4.8	⊙
Quartz gravel	60	Sand	34	0	6	45	5.3	■
Quartz gravel	60	Crushed rock	34	0	6	45	6.0	□
Quartz gravel	65	Sand	27.4	-[a]	5.6	45	3.5	+
Flint gravel	60	Sand	34	0	6	45	4.8	▲
Flint gravel	60	Crushed rock	34	0	6	45	6.9	△
Slag	60	Sand	33.7	0	6.3	45	5.0	▼
Slag	60	Crushed rock	33.7	0	6.3	45	5.6	▽
Slag	55	Sand	38.2	0	6.8	45	5.4	x

[a]2 percent cement.

Miner's rule for evaluating cumulative damage states that the condition at failure is given by

$$\sum_{i=1}^{r} \frac{n_i}{N_i} = 1 \tag{5}$$

where

n_i = number of cycles of stress σ_i applied to the test specimen, and
N_i = number of cycles to failure at constant stress amplitude σ_i from simple loading.

If N_c is the predicted fatigue life under variable amplitude conditions and $f_1, f_2, \ldots f_r$ are the relative proportions at each of the stress levels for which the fatigue lives are $N_1, N_2, \ldots N_r$ respectively, then by Miner's rule,

$$N_c = \frac{1}{\frac{f_1}{N_1} + \frac{f_2}{N_2} \cdots \frac{f_r}{N_r}} = \frac{1}{\sum_{i=1}^{r} \frac{f_1}{N_i}} \tag{6}$$

The application of this rule to the fatigue life of asphaltic mixes under compound loading conditions was considered by Monismith et al. (22), who suggested strain rather than stress to be the appropriate criterion. Although widely used in such design methods as employ load equivalency concepts, no experimantal justification of its use was available until 1965 when Deacon (23) published the results of an investigation into the behavior of asphaltic concrete under compound loading conditions. He concluded that the linear summation of cycle ratios governs the fatigue behavior of bituminous mixtures that are subjected to multiple strains of variable magnitude. That conclusion is supported by the results of a recent experimental investigation at Nottingham University (24). Variations in the level of strain during the course of a test were achieved either by changing the temperature of the specimen or altering the stress level.

In the application of Miner's rule, no allowance is made for intervals of "no-load" or rest periods. It may be assumed that the effect of such intervals will be beneficial or at worst of no consequence. If the effect is beneficial, then the rule, as it stands, is conservative for design purposes.

PREDICTION OF FATIGUE CHARACTERISTICS

To establish the fatigue characteristics of a particular asphaltic mix necessitates long and somewhat involved testing techniques using specially designed and expensive equipment. It would clearly be extremely useful if fatigue performance could be related to a test using more standard equipment, and the approach of Epps and Monismith (25) to try and correlate fatigue with a simple, tensile test seems valuable.

To date, there is little convincing evidence that a simple nonrepetitive, loading test will be able to predict fatigue performance over the wide range of conditions and materials necessary, particularly in view of the difficulty of defining service life. However, a simple test may possibly be used to investigate stiffness characteristics for a number of mix variables, and further research in this direction is warranted.

As more and more fatigue tests are carried out by various research workers in different parts of the world, the fund of results that may be used for design purposes grows. But unfortunately, different test methods and criteria of failure make it extremely difficult to correlate that fund of information in a quantitative manner.

The most promising approach at present appears to be for each agency to produce general laboratory relations for the more common mixes that are used in its location and correlate those with pavement performance in its environment. For example, Epps and Monismith (5) have suggested that the fatigue characteristics C and m in the basic strain-life relation shown in Eq. 4 should be 6.28×10^{-7} and 3.01 respectively for dense-graded aggregate mixes in California. These values give the expression

$$N_f = 6.28 \times 10^{-7}\left(\frac{1}{\epsilon}\right)^{3.01} \qquad (7)$$

Similarly, values of the characteristics C amd m obtained by Pell and Brown (13) and given in Table 1 are appropriate for some British mixes tested under particular conditions.

A recent development by several research workers is the production of empirical relations between mix-design variables and fatigue properties of asphalt mixes. That involves the identification of the most important parameters and establishing the influence of these on the fatigue relationship.

Kirk (26) states that the primary parameter is the stiffness of the binder and that the fatigue life under controlled stress flexure at a particular stiffness depends on the strain per binder volume. A factor is then applied that indicates improved performance with maximum size of aggregate, and another correction factor is applied that reflects the effect of binder and voids content.

Verstraeten (27) has produced a general expression based on controlled stress bending tests on 34 different mixes:

$$\epsilon(N) = \Phi \times C \times \frac{V_B}{V_B + V_V} \times N^{-0.22} \qquad (8)$$

where

$\epsilon(N)$ = initial strain to produce failure after N cycles;
Φ = coefficient depending on the asphaltene content in the bitumen;
C = coefficient that correlated with $V_A/(V_B + V_V)$;
V_A = volume of aggregate, percent;
V_B = volume of bitumen, percent; and
V_V = volume of voids, percent.

Equation 8 implies that the characteristic m, which defines the slope of the strain-life line, has a constant value of 4.5 for all mixes. Verstraeten applied his expression to selected results obtained by Pell and Taylor (4) for 22 further mixes and shows good agreement in all but a few cases.

It is doubtful whether in general a relation developed from the data of one researcher can be accurately applied to that of another. This is mainly because of the differences in apparatus and testing techniques, but an important fact that emerges from all fatigue investigations is that basically the general qualitative conclusions are similar.

The fatigue performance of a mix is generally expressed by the strain-life relation, and many workers have shown that over the practical range of fatigue life and tensile strain that relation is linear when plotted on a log-log basis and can be expressed as

$$\log N = \log C - m \log \epsilon \qquad (9)$$

Therefore, factors m and C characterize the fatigue performance of a particular mix. At Nottingham University, when 1 particular type of controlled stress testing was used, values of m and C were obtained for a wide variety of mixes having a comparatively large range of fatigue performances. It was found that there is a linear relation between m and log C (Fig. 13), which may be expressed in the form

$$m = A \log C + B \qquad (10)$$

That general relation only becomes apparent when mixes having a wide range of fatigue performances are considered and is probably not revealed by small changes in mix variables on a particular type of mix because of the normal scatter. Values of m and C obtained by other investigators show similar trends.

The relation between m and C indicates that the log strain-log life lines tend to radiate from a common intersection point, and, if either factor can be related to mix properties, then a simple method of fatigue performance prediction would result.

Multiple regression analyses on the mix variables involved showed that the most significant factors were binder viscosity, binder content, and void content. However, because the basic strain-life relations obtained from the controlled stress tests were found to be independent of temperature, an equiviscous measure, such as Ring and Ball temperature, was chosen to characterize the binder viscosity. Dobson (28) states that the temperature dependence of the viscoelastic properties of a bitumen may be described by one parameter, which Brodnyan (29) suggests is similar to the softening point.

Binder content and void content are, of course, closely related, and aggregate grading and state of compaction affect that relationship, so binder content by volume V_B and void content V_v were combined in a single factor $V_B/(V_B + V_v)$, i.e., ratio of binder volume to voids in the dry compacted aggregate.

Service life at a strain of 10^{-4} was used as a measure of fatigue performance, i.e., $N_s(\epsilon = 10^{-4})$, in a simple regression analysis that was performed on the results of 54 mixes and that gave the following relation:

$$\log N(\epsilon = 10^{-4}) = -16.34 + 6.03 \log\left(\frac{V_B \times 100}{V_B + V_v}\right) + 5.99 \log(T_{R\&B}) \tag{11}$$

the multiple correlation coefficient was 0.953, and the standard error of the estimate was 0.274. Values of measured fatigue life plotted against predicted values using Eq. 11 are shown in Figure 14. The service life at a strain of 10^{-4} was chosen for that analysis because in view of the inevitable scatter the results were considered more accurate at that strain level. However, either of the fatigue characteristic factors m or C could be related directly to the mix properties parameter by a similar approach, and further work is proceeding along those lines at the present time. In that case, the mix properties parameter consists of easily obtainable factors, namely, percentage volumes of binder and voids in the mix and the Ring and Ball temperature of the binder.

SUMMARY AND CONCLUSIONS

1. The definition of laboratory fatigue service life depends on the method of testing; therefore, it is not possible to produce unique relations characterizing fatigue performance.
2. Service life of specimens greatly depends on the mode of testing. Controlled stress loading appears to be applicable for materials used in thick asphalt construction, i.e., 6 in. or more, but controlled strain loading is more appropriate for thin asphalt layers, i.e., approximately 2 in. or less. Results from controlled stress mode of loading will generally give shorter lives and, hence, are conservative.
3. Whichever method of testing is used, the fatigue performance can best be characterized by a strain-life relation of the form

$$N_s = C\left(\frac{1}{\epsilon}\right)^m$$

where N_s is the number of applications of tensile strain ϵ to cause failure with the particular method of testing used. Factors C and m depend on the composition and properties of the mix and will also be affected by the testing method.

4. Mixes having maximum stiffness characteristics will give longer lives under controlled stress conditions. Therefore, for thick asphalt construction the choice of mix composition, namely, aggregate type and grading, filler content, and binder type and content, should be such that under compaction a mix of maximum tensile stiffness associated with minimum voids is obtained.
5. For thin asphalt construction maximum lives are likely to be obtained from more flexible, less stiff materials. Thus, mixes having high binder contents of softer bitumens should be used.

Figure 12. Fatigue lines for continuously graded dense asphalt mixes.

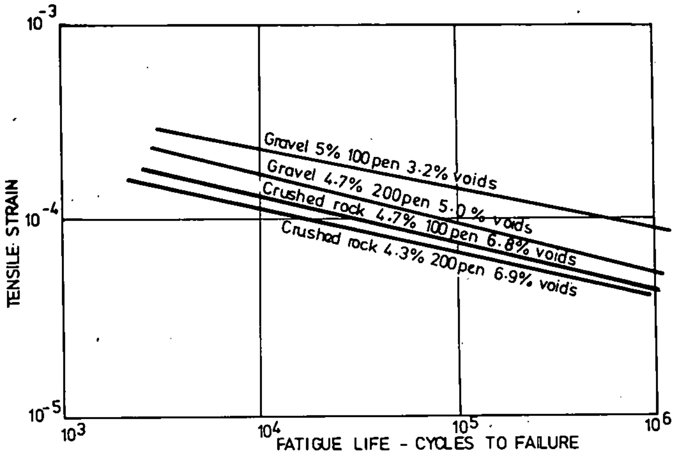


Figure 13. Relation between mix characteristic factors C and m.

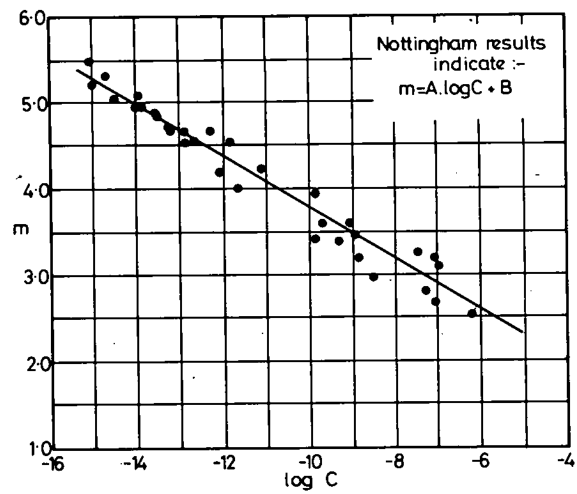
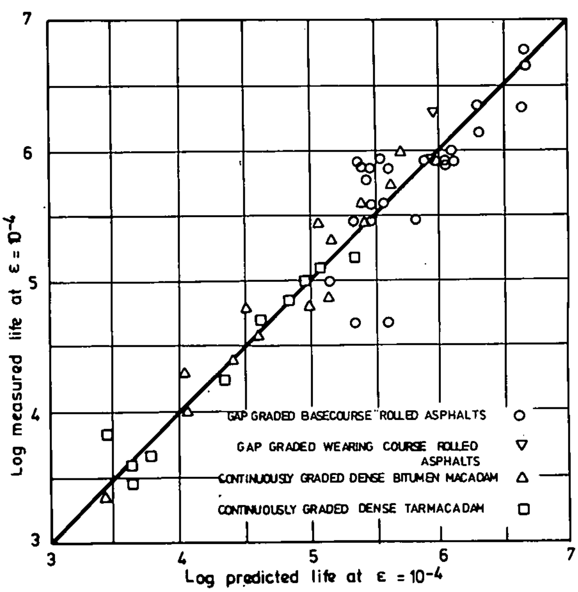


Figure 14. Measured versus predicted fatigue life.



6. A simple standard test probably using a single application of tensile load could usefully be developed to investigate, in a particular situation, the available mix compositions to give optimum stiffness characteristics.

7. General relations that have the form shown above in paragraph 3 and different values of mix characteristics C and m should be established by various agencies using their particular method of testing. Those relations can then be correlated with performance under the appropriate conditions of environment and traffic for design purposes.

8. The results of simple loading fatigue tests may be used for design in compound loading conditions by the use of Miner's rule, namely,

$$\sum_{i=1}^{r} \frac{n_i}{N_i} = 1$$

where

n_i = number of cycles of strain ϵ_1 applied, and
N_i = number of cycles to produce failure under constant strain amplitude ϵ_1 from simple loading tests.

The application of that rule is likely to yield conservative results for it neglects the beneficial effects of rest periods.

9. If it is not possible to characterize the material by fatigue testing, then use may be made of empirical relations relating the fatigue performance to mix properties. Various workers have produced relations that emphasize different mix parameters depending on the range of materials used to provide the data.

10. Because of the statistical nature of all fatigue results, it is important to realize that large numbers of specimens have to be tested before accurate relations can be established whether directly of the form shown above in paragraph 3 or indirectly based on previous results.

ACKNOWLEDGMENTS

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REFERENCES

1. Monismith, C. L. Asphalt Mixture Behavior in Repeated Flexure. Univ. of California, Berkeley, Rept. TE-66-6, 1966.
2. Monismith, C. L., and Deacon, J. A. Fatigue of Asphalt Paving Mixtures. Jour. Transp. Eng. Div., ASCE, No. TE2, 1969, pp. 317-346.
3. Pell, P. S., and Taylor, I. F. Fatigue Phenomena in Bituminous Systems. Proc., Int. Conf. on Struct., Solid Mech. and Eng. Des., Southampton, 1969, pp. 1103-1121.
4. Pell, P. S., and Taylor, I. F. Asphaltic Road Materials in Fatigue. Proc., AAPT, Vol. 38, 1969, pp. 371-422.
5. Epps, J. A., and Monismith, C. L. Influence of Mixture Variables on the Flexural Fatigue Properties of Asphalt Concrete. Proc., AAPT, Vol. 38, pp. 423-464.
6. Pell, P. S. Fatigue of Asphalt Pavement Mixes. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, 1967, pp. 577-593.
7. Majidzadeh, K., Kauffmann, E. M., and Ramsamooj, D. V. Application of Fracture Mechanics in the Analysis of Pavement Fatigue. Proc., AAPT, Vol. 40, 1971, pp. 227-246.
8. Deacon, J. A., and Monismith, C. L. Laboratory Flexural Fatigue Testing of Asphalt Concrete With Emphasis on Compound Loading Tests. Highway Research Record 158, 1967, pp. 1-31.

9. Kallas, B. F., and Puzinauskas, V. P. Flexural Fatigue Tests on Asphalt Paving Mixtures. ASTM Symp. on Fatigue of Compacted Bitumin. Aggregate Mixes, 1971.
10. Kirk, J. M. Results of Fatigue Tests on Different Types of Bituminous Mixes. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, 1967, pp. 571-575.
11. Bazin, P., and Saunier, J. Deformability, Fatigue, and Healing Properties of Asphalt Mixes. Proc., 2nd Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, 1967, pp. 553-569.
12. Van Dijk, W., Moreaud, H., Quedeville, A., and Uge, P. The Fatigue of Bitumen and Bituminous Mixes. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavemants, London, 1972, pp. 354-366.
13. Pell, P. S., and Brown, S. F. The Characteristics of Materials for the Design of Flexible Pavement Structures. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, London, 1972, pp. 326-342.
14. Raithby, K. D., and Sterling, A. B. The Effect of Rest Periods on the Fatigue Performance of a Hot-Rolled Asphalt Under Reversed Axial Loading. Proc., AAPT, Vol. 39, 1970, pp. 134-147.
15. Dehlen, G. L. The Effect of Nonlinear Material on the Behavior of Pavements Subjected to Traffic Loads. Univ. of California, Berkeley, PhD thesis, 1969.
16. Jiminez, R. A., and Gallaway, B. H. Behavior of Asphaltic Concrete Diaphragms to Repetitive Loadings. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, 1962, pp. 339-344.
17. Pell, P. S. Fatigue Characteristics of Bitumen and Bituminous Mixes. Proc., Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, 1962, pp. 310-323.
18. Moore, R. K., and Kennedy, T. W. Tensile Behavior of Asphalt-Treated Materials Under Repetitive Loading. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, London, 1972, pp. 263-276.
19. Santucci, L. E., and Schmidt, R. J. Effect of Asphalt Properties on the Fatigue Resistance of Asphalt Paving Mixtures. Proc., AAPT, Vol. 38, 1969, pp. 65-94.
20. Raithby, K. D., and Sterling, A. B. Some Effects of Loading History on Fatigue Performance of Rolled Asphalt. Gt. Brit. Transp. and Road Res. Lab., Crowthorne, Berkshire, Rept. LR496, 1972.
21. O'Neill, M. J. A Review of Some Cumulative Damage Theories. Australian Def. Sci. Serv. Aeronaut. Res. Lab., Struct. and Mater. Rept. 326, June 1970.
22. Monismith, C. L., Secor, K. E., and Blackmer, E. W. Asphalt Mixture Behavior in Repeated Flexure. Proc., AAPT, Vol. 30, 1961, pp. 188-222.
23. Deacon, J. A. Fatigue of Asphalt Concrete. Univ. of California, D Eng dissertation, 1965.
24. McElvaney, J. Fatigue of a Bituminous Mixture Under Compound Loading. Univ. of Nottingham, PhD thesis, 1972.
25. Epps, J. A., and Monismith, C. L. Influence of Mixture Variables on the Direct Tensile Properties of Asphalt Concrete. Proc., AAPT, Vol. 39, 1970, pp. 207-236.
26. Kirk, J. M. Relations Between Mix Design and Fatigue Properties of Asphaltic Concrete. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, London, 1972, pp. 241-247.
27. Verstraeten, J. Moduli and Critical Strains in Repeated Bending of Bituminous Mixes: Application to Pavement Design. Proc., 3rd Int. Conf. on Struct. Design of Asphalt Pavements, London, 1972, pp. 729-738.
28. Dobson, G. R. The Dynamic Mechanical Properties of Bitumen. Proc., AAPT, Vol. 38, 1969, pp. 123-135.
29. Brodnyan, J. G., Gaskins, F. H., Philippoff, W., and Thelen, E. Trans., Soc. Rheology, Vol. 4, 1960, p. 279.
30. Deacon, J. A. Fatigue Life Prediction. Paper in this Special Report.
31. Finn, F. N. Relation Between Cracking and Performance. Paper in this Special Report.
32. Majidzadeh, K., and Ramsamooj, D. V. Mechanistic Approach to the Solution of Cracking in Pavements. Paper in this Special Report.

Fatigue design is usually a subsystem of the overall pavement design system. Input variables to the fatigue design subsystem are load, environment, construction, structural, and maintenance. Usually construction, structural, and maintenance variables are controlled by the designer and basically affect the load-carrying capacity of the pavement as designed, constructed, and modified in service. Traffic and environmental variables impose a load on the pavement structure, and those are the ones considered in this paper. In the present state of the art, there are adequate methods for estimating traffic data. Environmental models can include a wide variety of inputs, but here the inputs are limited to moisture and temperature effects. Temperature can be predicted with some degree of adequacy, based on existing weather data, but no present method is completely adequate for predicting moisture effects in pavements although work is under way to improve techniques. Future improvements in pavement design systems in terms of traffic and environmental variables will necessarily involve some consideration of the stochastic variations in the predicted values, and the final designs will involve statistical confidence levels in lieu of conventional safety factors. However, adequate data are now available to develop rational fatigue design subsystems.

Other Input Variables: Traffic and Environmental

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In the past 10 years, a great deal of effort has gone into providing a better definition of the overall pavement design system (1, 2). In that system development, many charts and models have been drawn to depict the process of pavement management and design. They all have essentially the form shown in Figure 1, that is, a series of inputs combined through a model, usually mathematical, to generate a predicted response and a predicted history of loads to be transported. The response at some levels may yield distress, and all types of distress combine in some way, imprecisely known, to define performance in conjunction with the loads transported. Likewise, a series of concomitant variables is generated, resulting from the loads transported, the presence of the pavement structure, and other factors. A series of decision criteria and decision rules are applied to provide for rational design and management decisions.

Figure 1 shows that traffic and environment occur as both inputs and outputs to the pavement design process. Those inputs and outputs must not be confused, and the purpose of the paper is to discuss input variables to a fatigue design subsystem. As outlined by Chestnut (3), if the overall pavement design process is the system, then the fatigue design is a subsystem of that total design system.

INPUT VARIABLES

Input variables to the fatigue design subsystem can be listed in 5 categories: load, environment, construction, structural, and maintenance. The construction, structural, and maintenance variables are normally those that are controlled by the designer or pavement management team and that basically affect the load-carrying capacity of the pavement as designed, constructed, and modified in service. The traffic and environmental variables are usually those that impose a load on the pavement structure. Such a load may be induced by traffic or by the environment, i.e., a temperature stress. The construction, structural, and maintenance inputs will not be considered further in this paper because its purpose is to consider the traffic and environmental variables.

Interaction of Inputs

Figure 1 shows that the input variables to the design subsystem are not independent of each other but rather interact to the extent that the effect of one variable on the structural model depends on the level of one or more of the other input variables. That definition of interaction is important to the discussion that follows.

Feedback Information

The term feedback is used in pavement design in two ways. First, as shown in Figure 1, it relates to a change in the input variables that results from or "feeds back" from a change in the response or distress variables. Examples are (a) the increased loads induced by traffic when pavements get rough, or (b) the increased level of maintenance induced when pavements crack and must be repaired.

Another type of feedback is the information that is developed through observation and evaluation of pavement systems and their variables. Those data, in the form of traffic history, weather bureau data, deflection measurements, serviceability measurements, and condition surveys, constitute the feedback information necessary to improve design methods and adequately manage or maintain any particular pavement as well. Because of the inherent complexity of predicting traffic and environmental variables, measuring and recording such data including the effect of the variables on pavement performance provide the only realistic basis for developing improved design methods.

Much of the traffic and environmental input to a design subsystem such as fatigue must depend on recorded history of those variables or "feedbacks."

Variability

One other important concept is that of variability. All aspects of the pavement system involve inherent variations that cannot be measured adequately or predicted in the deterministic sense (4).

It is not possible to predict exactly the weight, amount, and kinds of traffic to be carried by the pavement at any given time during its lifetime. Likewise, it is impossible to predict exactly the temperature or weather conditions that will prevail for a particular design situation. As a result of such variability, pavement design subsystems must ultimately consider the stochastic variation of inputs, models, and model coefficients to predict a design in terms of required levels of reliability.

In the present state of the art, the problem is often one of reducing input variables to manageable proportions. For example, temperature is a continuously varying parameter in both space and time; thus, the modulus of the paving materials also varies constantly. Although it is necessary to estimate useful "design values" that can be used to simplify the design process, designers and researchers should continually work to find more appropriate ways to handle that complex variability.

TRAFFIC VARIABLES

Traffic variables include those related to the vehicle loads applied to the pavement and include total vehicle load; wheel load; tire pressure; wheel or gear configuration; lateral placement; volume or number of applications; lane and directional distributions; sequence of load applications of the various types; load type, i.e., static, dynamic, braking, or accelerating; and variability of traffic.

Traffic variables are among the most capricious with which the engineer must deal. Structural construction variables can be specified and controlled to some degree based on specifications and inspection. Environment cannot be controlled, but there are natural patterns that are generally repeated. Traffic, on the other hand, is a function of people, land use, legal load limits (although not constrained within them), and time. The number of variables that must be considered is large, and the number of combinations is infinite.

Must we then assume that the task of handling traffic is hopeless? No, not at all. There are many ways to attack the problem. They generally fall into 3 categories:

1. Design for the worst expected load condition,
2. Equate all loads to equivalent load applications, and
3. Predict traffic patterns and run a design analysis for the spectrum of load conditions to be encountered.

Predicting Traffic Input for Fatigue Analysis

In a fatigue analysis, it is not adequate to design for the so-called "worst" condition. Nor is it possible in the strictest sense to design for fatigue with equivalent loads, although the AASHO Road Test did develop equivalence values on the basis of equivalent accumulated damage (5). Subsequent design methods using that design concept (6) have used those equivalencies on the assumption that the mixed traffic relations developed by Scrivner et al. are adequate (7, 8).

As outlined by Scrivner, a major assumption required in the mixed traffic equations is that the order of accumulation of traffic is immaterial to the results. In the strictest sense, that is not true, of course. For example, it would be possible under that concept to apply many applications of a light load and then to apply a few overload applications to cause pavement failure. On the other hand, if the overloads were applied first and the pavement failed, it would not be possible to carry the large number of light loads. In practice, however, with a "regular" stream of traffic in which there is a rather random mixture, the assumption seems to work satisfactorily.

The Kentucky Highway Department uses equivalent 18-kip axle load in its design procedure (31). In that method, the expected lane loadings are determined from planning survey data, and all lanes are designed the same as the most heavily loaded lane. A cursory review of the work seems to indicate more concern with rutting than with fatigue, however.

The remaining approach to traffic consideration is to predict traffic as accurately as possible based on past history, projected land use changes and highway improvements, and predicted growth patterns. Each highway department has a technique for making such traffic projections based on data in its survey files. Derdeyn (9) and Heathington and Tutt (10) have studied this problem for Texas and have developed procedures that are acceptably reliable for use in the Texas flexible pavement system (1). Basically, the method includes 3 steps:

1. Count or estimate existing or currently predicted daily traffic by lane and direction at the site;
2. Estimate axle distribution based on comparisons with data from similar highways or from a data bank of statewide traffic; and
3. Predict future axle load distribution for the pavement site based on projected land use and population growth.

Darter has studied such typical traffic prediction methods and compared them with subsequent actual survey data (11). He points out in his report that uncertainties on the order of a factor of 2 can be expected in the data from any such predictions. Thus, if a traffic analysis predicts 10,000 applications, experience shows that the actual number of applications might prove to be as low as 5,000 or as high as 20,000.

To consider traffic adequately, the design method must ultimately include some consideration of uncertainty. Then it will be possible to evaluate the effect of improved traffic predictions in terms of high levels of design confidence and also to consider the resulting costs and benefits (4).

In the design analysis for the load spectrum, usually a series of layered system analyses is run on a computer by available programs. Those programs use a circular tire print, and the average tire pressure is usually used as the unit load. Multiple tires and axles can be handled by the more sophisticated programs. The designer must determine traffic data for his own design situation and input them into his method.

ENVIRONMENTAL VARIABLES

Environmental variables can include a wide variety of inputs such as general geological and soil conditions, oxygen and air surrounding the pavement, and moisture and temperature conditions within and around the pavement. However, for purposes of this discussion, we will limit our consideration of environmental variables to moisture and temperature effects.

It might be said that the pavement is a kind of transplant of materials into the existing body or environment of the earth for a special purpose. The environment works throughout the life of the pavement to "reject" it or destroy it by weathering. In other words, water and temperature variations are continually working on the pavement causing stress, strains, deflections, and permanent deformation, and there is much evidence that pavements are destroyed by the environment in the absence of traffic variables. In addition to moisture alone and temperature alone, there is, of course, the interaction of the 2 variables together in the form of freezing and thawing where the water and undesirable temperature variations result in water migration, formation of ice lenses or frost heaving, and subsequent softening in the spring.

The fatigue subsystem is primarily concerned with the effect of moisture and temperature on the structural properties or strength of the pavement materials or, in other words, the interaction of strength and material properties with the environment.

Moisture

The presence of moisture in a pavement produces several effects:

1. Oxidation of the surface in the presence of water and sunlight to change the properties of the material,
2. Freezing and thawing (formation of ice lenses),
3. Instability that is caused by excessive pore-water pressure and results in the failure of bases and slope stability,
4. Undesirable volume change, and
5. Stresses induced by moisture variation.

In this symposium, the main concern is the fatigue behavior of pavements. We will, therefore, ignore the volume change, roughness, and distress induced directly by moisture variations and will concentrate primarily on changes in material properties due to moisture variations and on stresses induced by soil-suction potentials present in the material or by excessive pore water pressure.

Moisture variations are extremely difficult to handle. They are sometimes considered in "regional factors," based in general on the average rainfall or the wet or dry condition of the geographical area in which the design is being considered. Such techniques are inherent in the Kansas method of pavement design (12). A similar concept was used in Texas (13); in it the general moisture conditions within the state were related to the "temperature constant."

A more precise and more desirable method of considering moisture variations in pavement design is to evaluate the soil-suction potential under the pavement. A great deal of work has been done with that concept, but most of it is related to the prediction of swell or shrinkage and not directly to the prediction of stresses. Measuring devices are being developed to accurately predict the soil-suction potential of a soil, but no really competent work has yet been done on relating that to fatigue.

Work by Lytton and Kher (14) presents an analytical tool for determining the changing moisture distribution with time in expansive clays. The method has been used to predict observed field values (Fig. 2, 14).

Figure 1. Typical pavement design system.

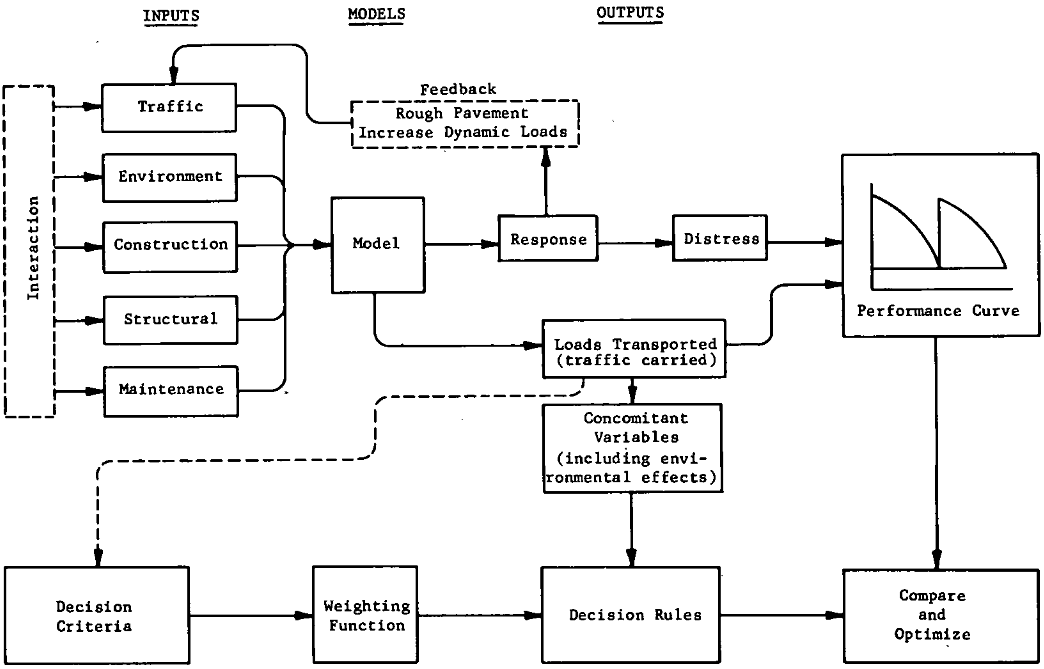
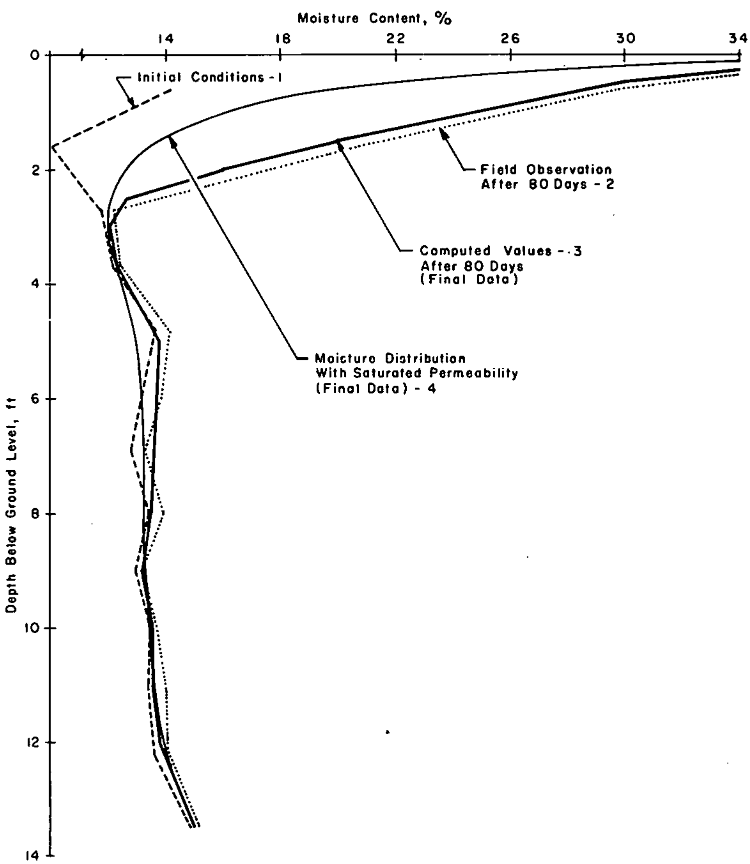


Figure 2. Moisture distribution study at tube 16.



Because moisture cycling is seasonal and not a daily change, moisture variations do not impose the large number of repeated stresses that temperature variations do. However, it is possible that significant changes in moisture content can result in severe pavement cracking. Work is beginning in west Texas at the present time to relate moisture-change stresses to pavement distress; however, no data are yet available.

In terms of evaluating strength for use in design evaluation, most designers evaluate feedback data from existing pavements in similar situations to determine the prevailing moisture conditions. In general, the water content often rises to near the liquid limit in many subgrades and stabilizes. Chu et al. (15) discuss a method of estimating moisture conditions based on laboratory tests.

Temperature

Of all environmental inputs, temperature is the easiest to estimate and use and is, therefore, used in more procedures than other environmental factors. Temperature stresses can often be as high as load stresses, as has been shown in numerous studies, particularly in rigid pavements, where temperature stresses are due to curling, warping, expansion, or contraction. Those same types of stresses are present in asphalt concrete pavements. They are tensile or compressive stresses due to increase or decrease in the general level of temperature and bending stresses due to temperature differential within the pavement structure itself.

The tensile or compressive stresses are due to general or seasonal changes in the level of temperature, as are the resulting changes in material properties, notably asphalt stiffness. The bending stresses, on the other hand, are due to diurnal or daily temperature cycles or variations. Much work has gone into the study of temperature in asphaltic concrete pavements. However, much remains to be learned in applying the resulting information to pavement design.

Temperature Stresses and Distress

In considering temperature, a number of authors have made significant contributions. Of particular note is work by Haas (16) and by Haas and Topper (17). That work refers to "low-temperature cracking" (Fig. 3). In reality, the effects are aggravated by low temperatures that tend to cause brittle mixes. The problem is equally applicable, however, to large changes in temperature, even though at higher levels. In west Texas, for example (18), a significant amount of temperature cracking has been observed and related to temperature variations.

Prediction of Temperature

Several approaches have been taken to the problem of predicting temperature input data.

1. General design adjustments have been made for "regional or environmental factors" (6, 19);
2. Design procedures have been developed that will relate to a general climatic area where an annual cycle of temperature, freezing, and so on will be about the same from year to year [that is the general approach taken by Haas and Topper, as shown by Figure 4 (16), in which "prior experience of cracking in the general area" is considered in the design]; and
3. Temperature profile and history of a particular pavement are predicted based on available weather data [significant work has been done in that area by Shahin and McCullough (18)].

Shahin and McCullough have developed a system for predicting daily temperature cycling during an average year. The system has the capability of simulating the temperature cycling at any depth from the surface of an asphalt concrete layer, assuming that the layer is semi-infinite. A model was developed by which asphalt

Figure 3. Factors of possible significance in low-temperature cracking of flexible pavements.

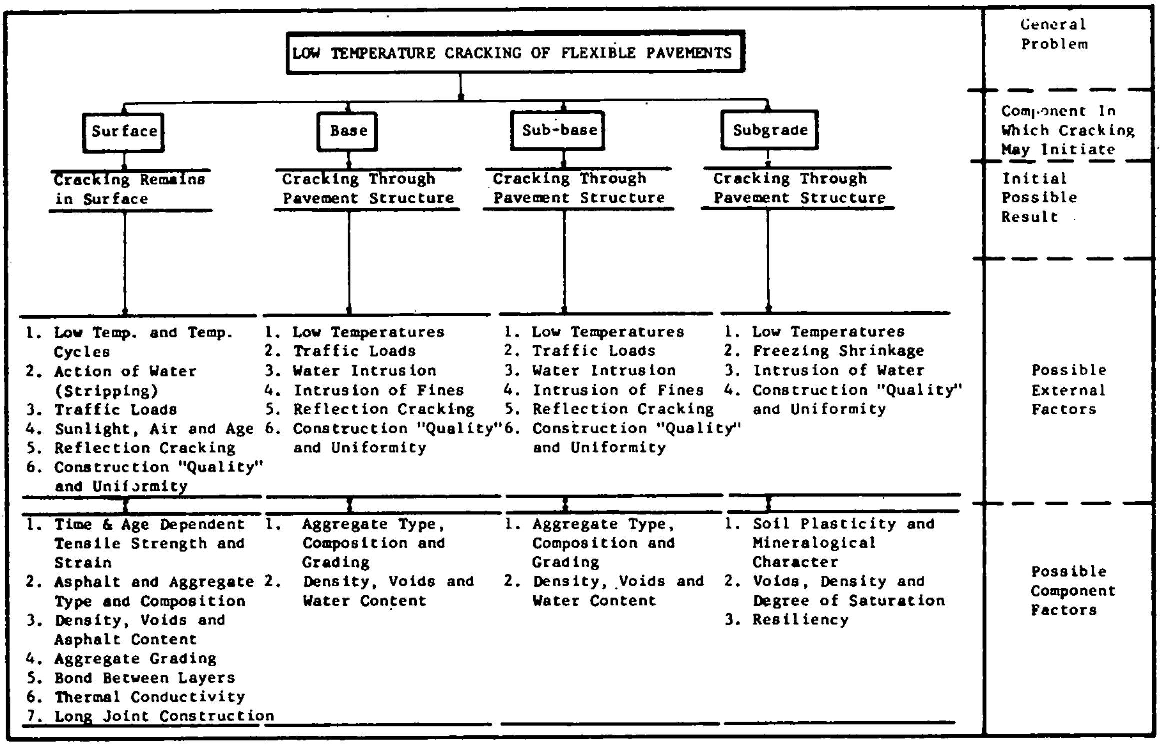
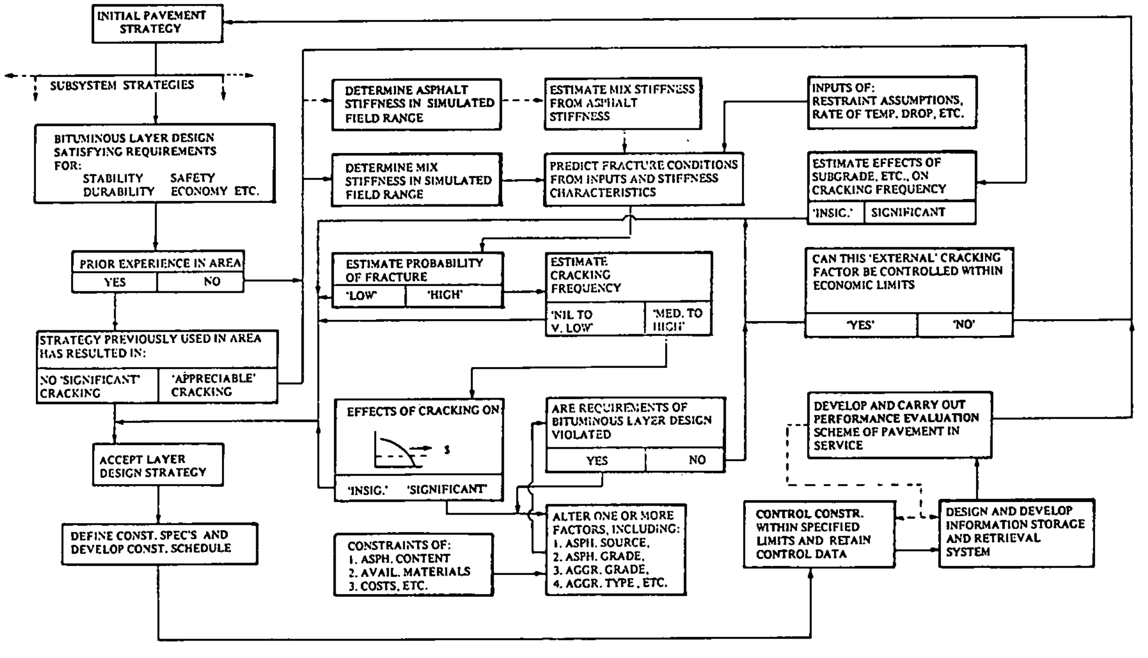


Figure 4. Tentative design and management subsystem for response of flexible pavements at low temperatures.



concrete temperatures during a single day can be predicted on an hourly basis. Figure 5 shows a comparison of temperatures predicted from the program and those measured (20).

The inputs to the model are daily mean air temperature, daily air temperature range, daily mean solar radiation, daily average wind velocity, and asphalt concrete thermal properties. The pavement temperature cycling during an average year is simulated by consideration of the day-to-day variations of its inputs. In doing so, Shahin and McCullough assume that the pavement thermal properties are constant. Meanwhile, they found that the daily mean air temperature and solar radiation are the most significant factors affecting pavement temperatures; therefore, equations to account for their variation were developed. From those equations, the daily mean air temperatures can be estimated for any day of the year, providing that the annual average and the range of air temperature are known. Similarly, the daily mean solar radiation can be estimated for any day of the year, providing that the July and annual averages of solar radiation are known. Figure 6 (18) shows a schematic diagram of the system.

Monismith et al. (21) developed a technique for predicting temperature distributions based on the heat-conduction equation:

$$TA(N) = ANNVE + (ANR/2)\ COS(N)$$

A typical temperature-time relation from their work is shown in Figure 7.

In general, methods such as those illustrated here can be used to predict the expected temperature-time history profiles for a particular design situation. Care must always be exercised, however, to ensure that such complex prediction models are compatible with reality. That can be done by a realistic comparison with measured temperature variations. Measured temperature variations in pavements can also be used to develop direct empirical temperature prediction equations.

Temperature Measurements

To evaluate temperature prediction models or to develop empirical temperature models requires some knowledge of the real temperature gradient present in a pavement. In 1966, Kallas (20) reported the results of a large series of temperature studies in thick asphalt concrete pavements. Results from those studies (Fig. 8 and Tables 1 and 2) can be used in a variety of ways to estimate temperatures for use in fatigue analysis or design.

Likewise, a large series of temperature measurements was taken at the AASHO Road Test. As shown in Figure 9 (5), deflection measurements on the pavements were highly correlated with the temperature or the time of year. Extremely low deflections were recorded for periods of frost. Because of the large installation of thermocouples, it was possible at the road test to develop isotherms for pavement sections, as shown in Figure 10 (5). An important part of the road test data involved the relation of pavement deflections and temperature. Typical information concerning that relation is shown in Figure 11 (5). Deflection-temperature relations basically involve changes in the material properties, i.e., stiffness, with changes in temperature. In all cases, decreased temperatures resulted in increased pavement stiffness and decreased deflection, as would be expected. The only variance with that relation occurred in the spring when the frost left the ground and moisture, which had accumulated during the freeze-thaw cycles, softened the subgrade and base materials to the point that greater deflections resulted. That relationship is supported by the observed deflections in the asphalt-stabilized base sections where the decreased temperatures could be expected to show low deflections, even though the subgrade was softened slightly by the accumulation of moisture. The observed data confirmed that hypothesis.

SUMMARY

In this brief report, we have considered 3 main categories of inputs to the pavement fatigue design subsystem: traffic, moisture, and temperature.

Figure 5. Measured and predicted pavement temperatures at College Park, Maryland, June 30, 1964.

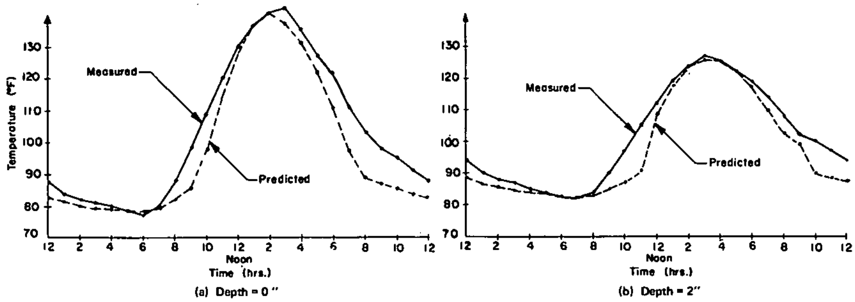


Figure 6. System for simulating asphalt concrete temperature cycling.

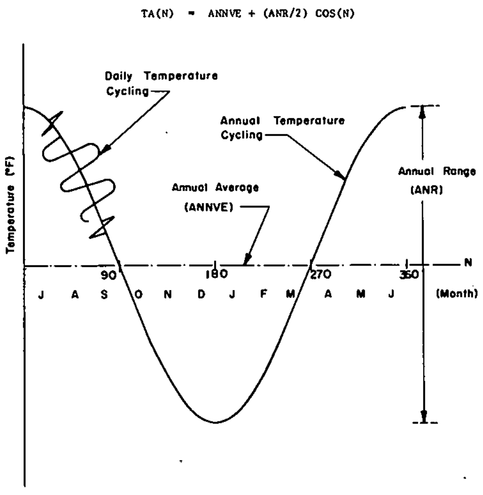


Figure 7. Temperature and time relation at various depths within slab for surface temperature variation of 0 to -40 F.

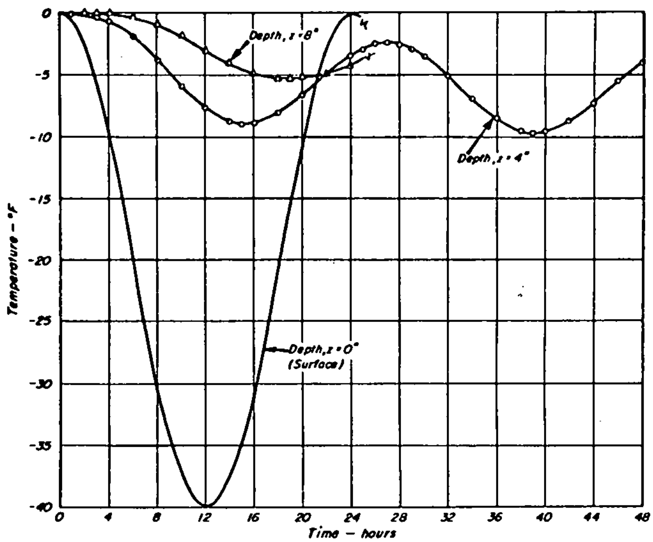


Figure 8. Asphalt concrete pavement temperatures on June 30, 1964.

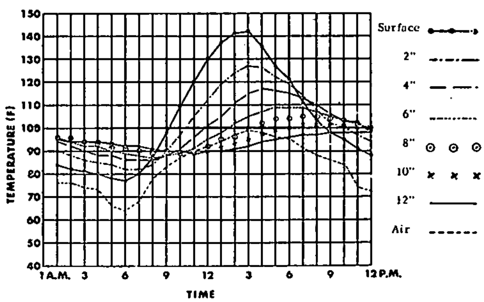


Table 1. Yearly duration of temperature levels of 12-in. asphalt concrete pavement.

Measurement Position	Duration (percentage of year) of Temperature Ranges (F): 0 to 9	10 to 19	20 to 29	30 to 39	40 to 49	50 to 59	60 to 69	70 to 79	80 to 89	90 to 99	100 to 109	110 to 119	120 to 129	130 to 139	140 to 149	Temperature Level (F): Avg	Avg High	Avg Low	High	Low
Air	-	2	7	15	19	17	17	14	7	2	-	-	-	-	-	54	65	41	99	2
Surface	-	1	5	11	17	15	13	14	9	5	4	3	2	1	-	64	87	48	142	9
2-in. depth	-	-	4	11	18	15	12	13	11	7	5	3	1	-	-	64	83	51	127	14
4-in. depth	-	-	2	11	18	17	12	11	14	9	5	1	-	-	-	64	74	55	119	18
6-in. depth	-	-	1	10	20	18	12	10	16	10	3	-	-	-	-	63	71	56	109	20
8-in. depth	-	-	1	9	21	18	12	9	18	11	1	-	-	-	-	64	69	58	105	23
10-in. depth	-	-	1	8	22	18	13	8	20	10	-	-	-	-	-	63	67	59	101	25
12-in. depth	-	-	-	7	24	17	13	8	22	9	-	-	-	-	-	64	66	60	98	27

Table 2. Monthly duration of temperature levels of soil below and adjacent to pavement.

Measurement Period	Measurement Position	Duration (percentage of month) of Temperature Ranges (F): 20 to 29	30 to 39	40 to 49	50 to 59	60 to 69	70 to 79	Temperature Level (F): Avg	Avg High	Avg Low	High	Low
November 1964	6-in. below 6-in. pavement	-	-	14	81	5	-	55	56	53	61	43
	12-in. depth in soil	-	1	35	64	-	-	50	52	48	56	38
	6-in. below 12-in. pavement	-	-	10	84	6	-	56	57	55	61	47
	18-in. depth in soil	-	-	27	73	-	-	51	52	50	56	42
December 1964	6-in. below 6-in. pavement	-	18	78	4	-	-	43	45	42	50	36
	12-in. depth in soil	-	43	57	-	-	-	40	42	38	49	33
	6-in. below 12-in. pavement	-	5	92	3	-	-	45	46	44	50	38
	18-in. depth in soil	-	20	80	-	-	-	42	43	41	48	36
January 1965	6-in. below 6-in. pavement	-	58	42	-	-	-	39	40	37	48	31
	12-in. depth in soil	-	80	20	-	-	-	35	36	34	45	30
	6-in. below 12-in. pavement	-	53	47	-	-	-	39	40	39	48	31
	18-in. depth in soil	-	67	33	-	-	-	37	37	36	45	31
February 1965	6-in. below 6-in. pavement	-	20	80	-	-	-	42	44	40	48	30
	12-in. depth in soil	-	51	49	-	-	-	39	40	37	46	30
	6-in. below 12-in. pavement	-	16	84	-	-	-	43	44	41	48	33
	18-in. depth in soil	-	38	62	-	-	-	39	40	39	46	33
March 1965	6-in. below 6-in. pavement	-	1	87	12	-	-	48	50	45	56	39
	12-in. depth in soil	-	11	88	1	-	-	43	44	40	50	36
	6-in. below 12-in. pavement	-	-	82	18	-	-	48	49	46	54	42
	18-in. depth in soil	-	1	99	-	-	-	43	44	42	48	38
April 1965	6-in. below 6-in. pavement	-	-	6	74	20	-	56	59	53	67	45
	12-in. depth in soil	-	-	43	57	-	-	49	51	47	59	40
	12-in. pavement	-	-	3	78	19	-	56	58	54	64	48
	18-in. depth in soil	-	-	45	55	-	-	49	50	48	56	42

Figure 9. Seasonal deflection on nontraffic loop, 6-kip single axle load.

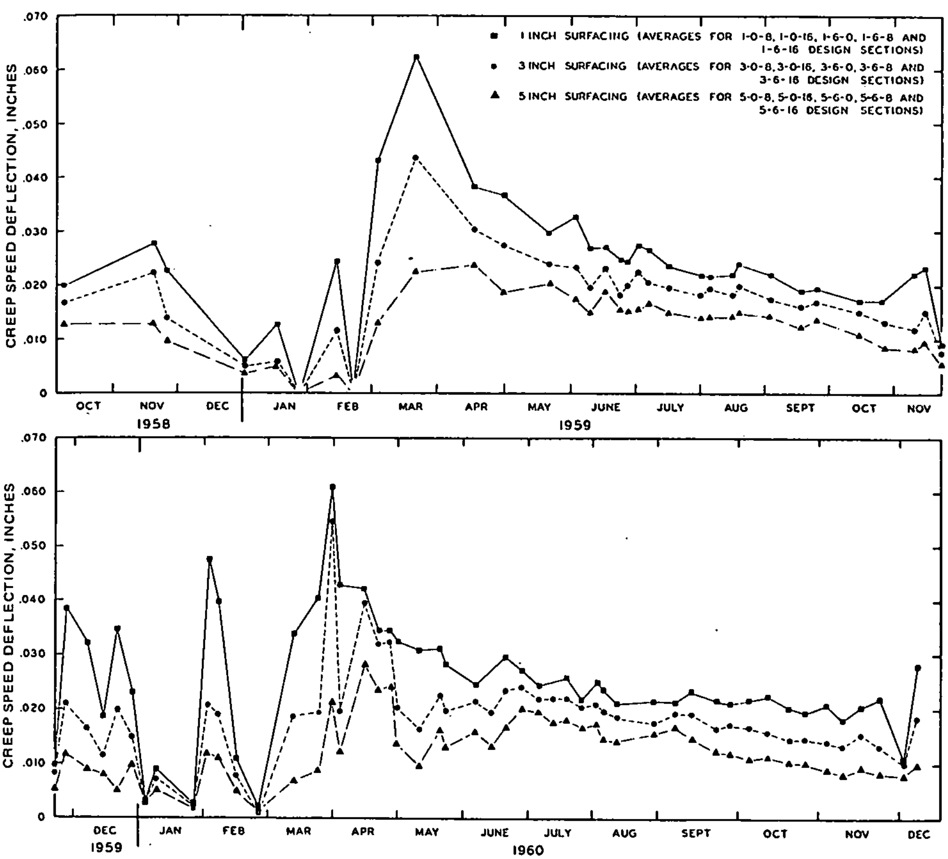


Figure 10. Isotherms (3-0-16 design) of loop 1 for March 25, 1960 (left), and May 25, 1960 (right).

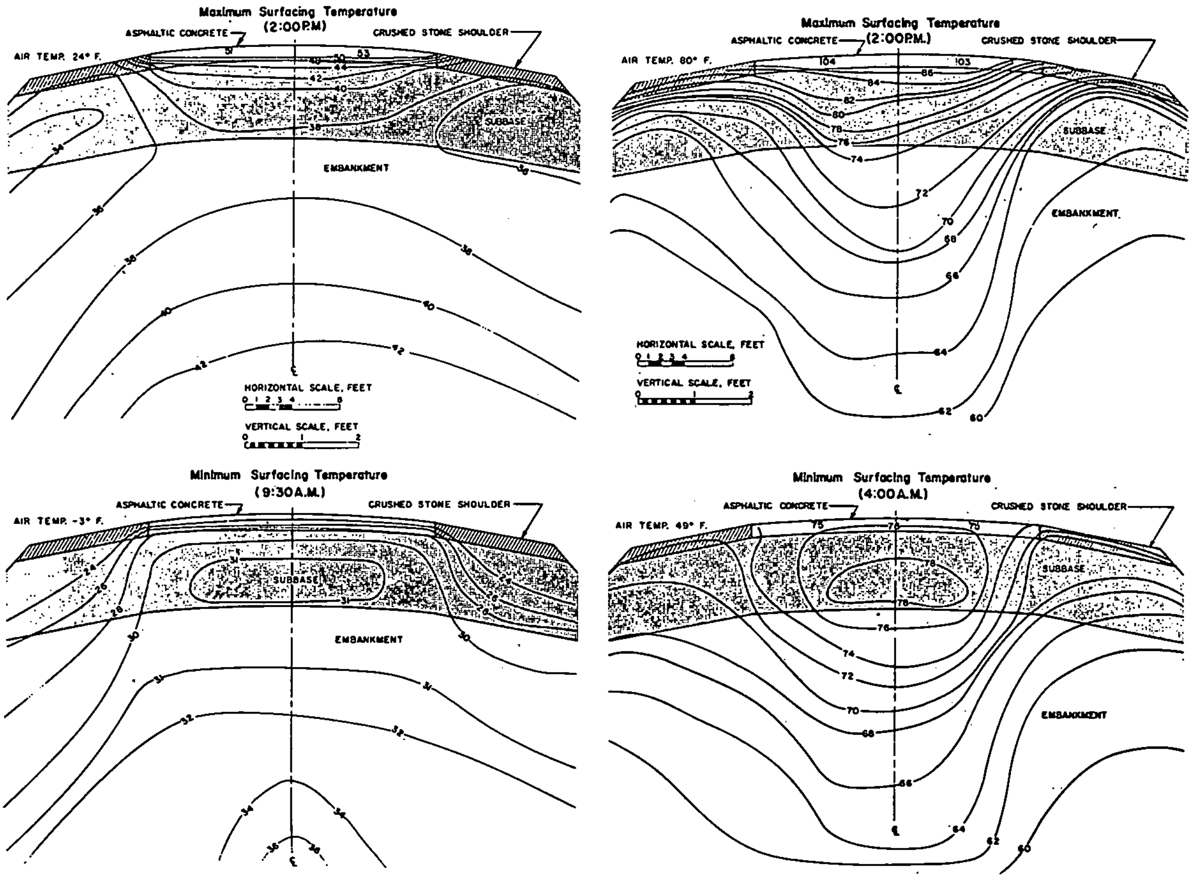
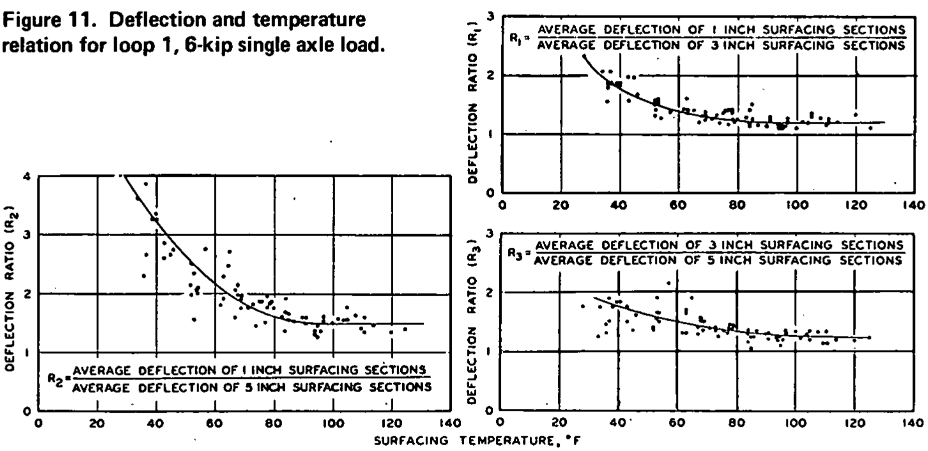


Figure 11. Deflection and temperature relation for loop 1, 6-kip single axle load.



In the present state of the art, there are adequate methods for estimating traffic data based on past experience and prediction models. Temperature can also be predicted with some degree of adequacy based on existing weather data. Moisture is perhaps the hardest to evaluate. No present method is completely adequate to predict moisture effects in the pavement, although work is under way to improve the techniques.

Future improvements in pavement design systems in terms of traffic and environmental variables will of necessity involve some consideration of the stochastic variations in the predicted values. Thus, the final designs will probably involve statistical confidence levels in lieu of conventional safety factors. However, adequate data are now available to develop rational fatigue design subsystems.

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REFERENCES

1. Hudson, W. R., McCullough, B. F., Scrivner, F. H., and Brown, J. L. A Systems Approach Applied to Pavement Design and Research. Texas Highw. Dept.; Texas Transp. Inst., Texas A&M Univ.; and Cent. for Highw. Res., Univ. of Texas at Austin, Res. Rept. 123-1, March 1970.
2. Hudson, W. R., and McCullough, B. F. Development of SAMP: An Operational Pavement Design System. Materials Research and Development, Inc., Oakland, Calif., Final rept. of NCHRP Proj. 1-10, Dec. 1970.
3. Chestnut, B. H. Systems Engineering Tools. John Wiley and Sons, New York, 1965.
4. Darter, M. I., McCullough, B. F., and Brown, J. L. Reliability Concepts Applied to the Texas Flexible Pavement System. Highway Research Record 407, 1972, pp. 146-161.
5. The AASHO Road Test: Report 5—Pavement Research. HRB Spec. Rept. 61E, 1962.
6. Langsner, G., Huff, T. S., and Liddle, W. J. Use of Road Test Findings by AASHO Design Committee. HRB Spec. Rept. 73, 1962, pp. 399-414.
7. The AASHO Road Test. HRB Spec. Rept. 73, 1962.
8. Scrivner, F. H. A Theory for Transforming AASHO Road Test Pavement Performance Equation to Equations Involving Mixed Traffic. HRB Spec. Rept. 66, 1961, pp. 39-46.
9. Derdeyn, C. J. A New Method of Traffic Evaluation for Pavement Design. Highway Research Record 46, 1964, pp. 1-10.
10. Heathington, K. W., and Tutt, P. R. Estimating the Distribution of Axle Weights for Selected Parameters. Highway Research Record 189, 1967, pp. 44-78.
11. Darter, M. I. Uncertainty Associated With Predicting 18-Kip Equivalent Single Axles for Texas Pavement Design Purposes. Cent. for Highw. Res., Univ. of Texas at Austin, Oct. 1971.
12. McCullough, B. F., Van Til, C. J., Vallerga, B. A., and Hicks, R. G. Evaluation of AASHO Interim Guides for Design of Pavement Structures. Materials Research and Development, Inc., Oakland, Calif., Final rept. of NCHRP Proj. 1-11, Dec. 1968.
13. Scrivner, F. H., Moore, W. M., and Carey, G. R. A Systems Approach to the Flexible Pavement Design Problem. Texas Transp. Inst., Texas A&M Univ., Res. Rept. 32-11, 1968.
14. Lytton, R. L., and Kher, R. K. Prediction of Moisture Movement in Expansive Clays. Cent. for Highw. Res., Univ. of Texas at Austin, Res. Rept. 118-3, May 1970.

15. Chu, T. Y., Humphries, W. K., and Sphen, S. N. A Study of Subgrade Moisture Conditions in Connection With the Design of Flexible Pavement Structure. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, Vol. 1, Sept. 1972.
16. Haas, R. C. G. Thermal Shrinkage Cracking of Some Ontario Pavements. Ontario Dept. of Highw., Rept. RR161, May 1969.
17. Haas, R. C. G., and Topper, T. H. Thermal Fracture Phenomena in Bituminous Surfaces. HRB Spec. Rept. 101, 1969, pp. 136-153.
18. Shahin, M. Y., and McCullough, B. F. Prediction of Low-Temperature and Thermal-Fatigue Cracking. Texas Highw. Dept.; Texas Transp. Inst., Texas A&M Univ.; and Cent. for Highw. Res., Univ. of Texas at Austin, Res. Rept. 123-14, 1973.
19. Committee on Design. AASHO Interim Guide for the Design of Flexible Pavement Structures. AASHO, 1961.
20. Kallas, B. F. Asphalt Pavement Temperatures. Highway Research Record 150, 1966, pp. 1-11.
21. Monismith, C. L., Secor, G. A., and Secor, K. E. Temperature Induced Stresses and Deformation in Asphalt Concrete. Proc., AAPT, Vol. 34, 1965.
22. Hudson, W. R., and Kennedy, T. W. The Airfield Pavement System and Its Parameters. Austin Research Engineers, Inc., Texas, March 1970.
23. Haas, R. C. G., et al. Low-Temperature Pavement Cracking in Canada: The Problem and Its Treatment. Canadian Good Roads Assn., Montreal, 1970.
24. Lytton, R. L. Theory of Moisture Movement in Expansive Clays. Cent. for Highw. Res., Univ. of Texas at Austin, Res. Rept. 118-1, Sept. 1969.
25. Hutchinson, B. G., and Haas, R. C. G. A Systems Analysis of the Highway Pavement Design Process. Highway Research Record 239, 1968, pp. 1-24.
26. Anderson, K. O., Shields, B. P., and Dacyszyn, J. M. Cracking of Asphalt Pavements Due to Thermal Effects. Proc., AAPT, Vol. 35, Feb. 1966, pp. 247-262.
27. Breen, J. J., and Stevens, J. E. Fatigue and Tensile Characteristics of Bituminous Pavements at Low Temperatures. Sch. of Eng., Univ. of Connecticut, Rept. JHR-66-3, July 1966.
28. Nontraffic Load Associated Cracking of Asphalt Pavements. Proc., AAPT, Vol. 35, Feb. 1966, pp. 239-357.
29. Haas, R. C. G. The Performance and Behavior of Flexible Pavements at Low Temperatures. Univ. of Waterloo, Ontario, PhD thesis, June 1968.
30. Monismith, C. L., Alexander, R. L., and Secor, K. E. Rheologic Behavior of Asphalt Concrete. Proc., AAPT, Vol. 35, Feb. 1966, pp. 400-450.
31. Havens, J. H., Deen, R. C., and Southgate, H. F. Pavement Design Schema. Paper in this Special Report.

That bituminous highway and airfield pavements can and should be designed in part to control fatigue distress is no longer questionable. This state-of-the-art paper describes how such a design can be accomplished by means of the phenomenological approach and emphasizes those procedures essential to the prediction of fatigue life. Each element of the design process is presented in terms of a comprehensive flow chart that depicts not only the interrelations among individual elements but also the chronological sequence within which the individual elements are addressed. Critical stresses and strains in the pavement structure can be estimated sufficiently well by means of elastic, multilayered analysis. That analysis is made tractable only by approximating the continuous spectra of traffic loads and physical states of the pavement by means of discrete categories. Failure criteria relating the number of load applications causing failure to the calculated strain level and other variables are used to estimate the fatigue damage caused by 1 application of each traffic load while the pavement is in each physical state. The total accumulation of fatigue damage during the design life is estimated by the hypothesis of the linear summation of cycle ratios (Miner's hypothesis). Failure criteria for use in routine design can probably best be developed from analyses of the performance of in-service pavements. However, methods of structural analysis, materials characterization, and pavement-state categorization must be identical during both development of the criteria and their application to design. Proper selection of the physical-state categories affords an excellent opportunity for realistically recognizing the effects of environmental variables on pavement performance and design.

Fatigue Life Prediction

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The one structural function of a highway or airport pavement is to ensure that stresses repetitively applied to the subgrade by traffic loads are sufficiently small that shear failure and excessive permanent deformation within the subgrade are prevented. At the same time, the structure itself must be so constructed as to prevent internal distress that might ultimately impair the ability of the pavement to carry traffic safely and smoothly. One of several types of internal distress that must be controlled is fatigue, that is, cracking of the bound components of the pavement due to repetitive traffic loading.

The notion that pavement structures can exhibit fatigue distress and that such distress can and should be controlled by judicious design is certainly not new. As early as 1938, Bradbury (3) suggested a design methodology for controlling fatigue distress in portland cement concrete pavements. The central thrust of the current Portland Cement Association design methodology (34) is the control of fatigue cracking through proper thickness selection. Design techniques have also been advanced for controlling fatigue in bituminous pavements. In 1963, Shell made a significant contribution to available technology through the introduction of a design methodology for highway pavements (36). That was followed in 1970 (12) by the development of a design procedure for bituminous airfield pavements. The Kentucky Department of Highways has developed a comprehensive design methodology (8, 9) that, like the Shell procedures, incorporates a concern with fatigue cracking.

The Asphalt Institute has also incorporated means for controlling fatigue distress in its new manual (1) for the design of full-depth asphalt pavements for airfields.

In addition to those methodologies that have immediate applicability to routine design situations (41, 42), there have been several notable research investigations in which the prevention of fatigue cracking has been examined in detail (27, 40). The purpose of this paper is to summarize some of the currently available techniques necessary for fatigue-life estimation in bituminous pavements.

DESIGN APPROACH

The recommended approach for design of bituminous pavements to control fatigue cracking is similar to that for the design of most engineering structures. Basically it is a trial-and-error procedure whereby (a) a trial structure is assumed, (b) the structure is analyzed by estimating the levels of the critical stresses and strains anticipated under in-service loading, (c) the structure is evaluated by comparing the estimated stresses and strains with tolerable levels derived from failure criteria, and (d) modifications are made to the trial structure as necessary, and the process is repeated until a satisfactory design has evolved.

A more detailed representation of the design process as it can be applied to the control of fatigue cracking in bituminous pavements is shown in Figure 1. The process flow is described below.

1. Select the design life in order to enable an estimate of traffic accumulation (Fig. 1, box 24).
2. Select or specify materials to be used in the trial pavement from among those available locally (box 21).
3. Select a trial structural section, that is, the thicknesses of the component layers (box 22).
4. Select the physical states of the pavement structure that are anticipated in service and that are to be used in the design process (box 23). A physical state is that which corresponds to given moisture and temperature profiles or an equivalent representation thereof. If the physical states of the pavement are properly defined, regional effects as influenced by climatic conditions may be properly considered in the design.
5. Estimate the future traffic that is anticipated while the pavement is within each distinct physical state during the entire design life (box 52).
6. Characterize the deformability, that is, the stress-strain response, of the trial materials either by direct laboratory testing (box 41) or by comparisons with prior testing of similar materials (box 33).
7. Estimate the levels of critical stresses or strains or both in the pavement structure that result from the application of each traffic load while the pavement is in each physical state (box 51).
8. Establish simple failure criteria, that is, the number of load applications causing fatigue failure for each of all possible combinations of traffic loads and pavement states (box 50). That is a somewhat complex process and may entail any or all of the following: laboratory fatigue testing of bituminous materials (box 40), comparisons with prior laboratory fatigue testing of similar bituminous materials (box 32), evaluation of failure criteria developed by others (box 31), analysis of the performance of in-service pavements (box 30), and selection of a suitable level of terminal serviceability, that is, definition of failure (box 20).
9. Estimate the accumulation of fatigue damage in the trial pavement (box 60) under the anticipated design loading (box 52) by suitably comparing the estimated history of stress or strain or both (box 51) with the simple failure criteria (box 50).
10. Determine the acceptability and optimality of the trial pavement and, if necessary, modify the materials or structure or both of the trial pavement and iterate (boxes 70 and 71).
11. Examine all other failure modes (box 80).

That design process may be viewed by pavement designers as excessively long and complex when it is compared with many of the more conventional procedures. However, after the methodology has been developed, routine designs can be accomplished with ease. Simple charts or nomographs can often be used to depict design relations, and, if desired, laboratory and field testing can be minimized.

DESIGN DETAILS

In many respects, bituminous pavements are extremely complex structures. That is partly due to the extensive influence of the climatic environment on pavement response to traffic loading. The stiffness of the bituminous layers is strongly dependent on pavement temperature. Also the mechanical response of many subgrade soils is influenced by prevailing moisture and temperature conditions. During its life in service, a pavement undergoes continual fluctuation in its temperature and moisture profiles. Analyses of pavement structures, such as those described here, are made tractable only by classifying all environmental conditions into a limited number of discrete categories or states. The subscript j will be used hereinafter to represent the jth physical state of the pavement that corresponds to a particular set of moisture and temperature conditions within the structure.

In a similar way, the continuum of traffic loads is best treated in analyses such as these by all loads being classified into a limited number of discrete categories. For highway vehicles, that classification is normally based on axle type, for example, single or tandem, and on axle load. The subscript i will be used hereinafter to represent a traffic load of the ith type or category.

Analysis of fatigue is based on the concept that each load application induces in the pavement irrecoverable damage and that, under load repetitions, such damage continually accumulates until such time that the pavement fails, that is, is no longer serviceable. In the phenomenological approach to fatigue analysis, damage is represented by a simple fraction (or percentage). The pavement is said to fail in fatigue when the cumulative damage under repetitive loading reaches 1.0 (or 100 percent). Failure may be defined in many ways, such as the point of fatigue-crack initiation or the point at which cracking has progressed to a given extent. In any case, the damage at failure is considered to be complete, that is, 100 percent.

Immediately following are discussions of some of the details of fatigue analysis and design. For clarity of presentation, the discussion proceeds in reverse numerical order with respect to the design sequence shown in Figure 1.

Evaluation and Decision

The evaluation phase consists of fatigue life estimation, that is, estimation of the cumulative damage that is induced in the trial pavement by the loading anticipated during the design life (Fig. 1, box 60). A structurally efficient pavement is one in which the cumulative damage during the design life is estimated to be exactly 1.0. Normally, in analyses of bituminous pavements, no allowance is made for a factor of safety.

Let d_{ij} be the damage induced in the pavement by 1 application of the ith load while the pavement is in the jth physical state. If that particular load is repetitively applied to the pavement in that state until the pavement fails and if N_{ij} represents the number of applications before failure, then d_{ij} can be estimated in the most simple way as follows:

$$d_{ij} = \frac{1}{N_{ij}} \tag{1}$$

Because both the magnitude of traffic loads and the physical state of the pavement continuously vary for in-service pavements, N_{ij} cannot be measured directly. Hence, it is an estimated or a derived quantity that is usually related to the level of applied strain ϵ_{ij}.

The traffic forecast (Fig. 1, box 52) yields an estimation of n_{ij}, the predicted number of applications of load i on the pavement in state j during the design life. The total cumulative damage D predicted during the design life is then

$$D = \sum_j \sum_i d_{ij} n_{ij} = \sum_j \sum_i \frac{n_{ij}}{N_{ij}} \tag{2}$$

[Equation 2 must be slightly modified when it is necessary to consider the transverse distribution of traffic loads across the pavement, that is, the degree of channelization. Such a consideration is thought to be unnecessary for highways but is an absolute necessity for airfield pavements (5, 39)]. Failure occurs if D equals or exceeds 1.0. Equation 2 represents the well-known hypothesis of linear summation of cycle ratios, also called Miner's hypothesis. It is the basis for the current design procedures of the Portland Cement Association (34) and has routinely been used in the design of aircraft structures (15). More important, all current design procedures for bituminous pavements that are based on the phenomenological approach use Miner's hypothesis. A limited laboratory investigation of bituminous materials has indicated the validity of Miner's hypothesis under randomly varied loads when the physical state is held constant (7). Recent laboratory work at the University of Nottingham confirms the validity of that hypothesis when the physical state is varied (43).

Equation 2 is used to estimate the damage anticipated in the trial pavement during the design life. If the damage exceeds one, the pavement is underdesigned, and the trial structure or materials or both must be modified and the process repeated (box 71). If the damage equals one or is slightly less than one, the trial pavement is accepted as a suitable design (box 70). If the damage is considerably less than one, the pavement is overdesigned and modification of the trial structure and iteration are required (box 71).

Analysis

The purpose of the evaluation phase is simply to estimate how much fatigue damage is expected to accumulate in the trial pavement during the design life. Input to that evaluation is generated in the analysis routines and consists simply of the volumes of traffic anticipated during the design life, n_{ij}, and estimates of the number of repetitions of each traffic load that would cause fatigue failure if that load were repetitively applied in the absence of other loads, N_{ij}. The usual procedure for estimating N_{ij} is (a) establish a failure criterion that relates the number of repetitions to failure N with some critical strain ϵ induced in the pavement and perhaps with other variables such as temperature, (b) estimate the critical strains ϵ_{ij} in the pavement induced by various loads under different physical states by means of analytical simulation, and (c) determine N_{ij} by combining the output of the 2 prior analyses. That procedure is summarized in subsequent sections after a brief discussion of traffic estimation.

Future Traffic (Box 52)

Obtaining an estimate of n_{ij} can be tedious and time-consuming, but it is one of the most critical portions of the fatigue-design process. Detailed procedures are described elsewhere (44), and it is only necessary here to emphasize a few relevant points.

Design is usually based on conditions at a critical location on a highway or airfield. For multilane highways, that is normally the outside lane that carries larger volumes of damaging truck traffic. There may also be a significant difference in the direction of travel along a particular route. Thus, detailed information is necessary on lane and directional distributions of the highway traffic. For airfields, the critical location is most frequently a taxiway servicing the most heavily used runways. In any case, the estimate must be based only on that volume and character of traffic traversing the critical or design location.

Design for fatigue can be accomplished equally well by treating the basic loading patterns directly or by expressing the relative destructive effects of different loads and configurations in terms of repetitive equivalency factors. Considerable effort, both empirical and theoretical, has been directed to the establishment of applicable load equivalency factors for highways (6, 21, 29, 37) and airports (5, 39). Expressing the destructive effects of a wide spectrum of traffic loads in terms of equivalent repetitions of a standard or base load greatly simplifies the analytical process (box 51) and enables simplified presentation of design relations in a conventional manner.

It is important to recognize that traffic estimates must be related to the physical states of the pavement. If the physical state is represented by average annual conditions, that is a simple task. However, if other more precise representations of the physical states are used, then typical monthly and perhaps even daily fluctuations in traffic volumes must be recognized (27).

Care must be taken to avoid a priori assumptions as to the potential significance of certain traffic loads, for example, the steering axles of trucks and the nose gears of aircraft. For identical loads, a steering axle (single tires) has been found to be approximately 3 times more destructive in fatigue than a conventional single axle (dual tires) (6). Although the total loads on steering axles are usually smaller than those on other axles, steering axles can potentially contribute in a significant way to the total accumulation of fatigue damage.

Procedures for estimating n_{ij} are no different from those required in many current design methodologies with the exception that the volumes may need to be classified according to the physical states of the pavement. Other traffic information may be required, however, including items such as speed, tire inflation pressures, and average distances separating various tires and axles.

Computation of Critical Strains (Box 51)

After the traffic loads, that is, the categories designated by i, have been identified, it is necessary to estimate the critical strains ϵ_{ij} in the pavement. Those strains are combined with the failure criterion (box 50) to obtain estimates of N_{ij}, the remaining set of unknowns in Eq. 2.

Critical Strain

Under conditions of constant air-void and asphalt contents, the determinant of fatigue failure is the maximum principal tensile strain (32). When the bituminous-bound layer of a pavement behaves as a structural slab and is of sufficient thickness, that strain occurs at the underside of the bound layer. Therefore, for normal pavements, the critical strain for fatigue design is the maximum principal tensile strain at the bottom of the bound layer.

Method of Structural Analysis

Methods for the structural analysis of layered systems are described elsewhere (45). The simplest of those, in which the pavement is represented by a linearly elastic, multilayered structure of semi-infinite extent, has been found to reasonably approximate the behavior of bituminous pavements (16) and is readily available in the form of computer software (33, 38) or graphical representation (18, 31). To compute the maximum principal tensile strain in the bound layer requires that each layer of the pavement be characterized by its thickness, modulus of elasticity, and Poisson's ratio.

A structure consisting of bituminous surface and base courses, unbound granular base, and subgrade is usually characterized as a 3-layer system. If there is a thermal gradient in the bound layers, the average temperature is used to derive a suitable modulus of elasticity to represent the composite behavior of the bound layers. Likewise, average moduli may be used to represent the behavior of the base and subgrade layers. A more refined analysis is made possible by representing that same structure by more than 3 layers. That results in a more accurate estimation of stresses and strains at the cost of added analytical complexity. For purposes of routine design, a 3-layer representation is thought to be sufficiently accurate.

The moduli of some materials, most notably unbound granular bases, has been found to depend on the level of applied stress. Unfortunately, the level of applied stress is unknown before the analysis. To consider such a situation, one may use an iterative solution of the multilayered elastic system (26). Such a refinement is highly desirable for research, investigative, and developmental purposes but may be of lesser consequence for routine design procedures.

Critical tensile strains ϵ_{ij} must be computed for a relatively large number of traffic loads i when applied to the trial pavement under a relatively large number of physical states j. That can lead to a formidable number of computations. The burden can be eased substantially by the use of repetitive load equivalency factors that make it necessary to compute only those strains resulting from the standard or base load. Another means for expediting the computations is to make extensive use of interpolation techniques (5). Finally, regression techniques may be used to expand a limited number of computed strains to a much more extensive set. Witczak quite successfully used such a technique in his analysis of 2-layered, airfield structures (39).

Traffic Loading

Readily available computer solutions of the elastic, multilayer system employ a tire load that has a circular contact area and a uniform contact pressure and is applied in a direction normal to the pavement surface. The contact pressure is most often assumed to equal the hot, tire-inflation pressure. Impact loading is seldom considered in the design.

Multiple tires in close geometrical proximity can be accurately represented in the analysis. However, a common practice is to approximate the effects of multiple tires through the use of an equivalent single tire. That and similar procedures can lead to erroneous results (6) and should not be used without independent validation.

Establishment of Failure Criterion (Box 50)

The failure criterion is simply a relation between the maximum principal tensile strain ϵ in the bound layers of the pavement and the number of applications N of that particular strain level causing failure by fatigue. (The failure criterion describes the situation that would result if only one strain level were repetitively applied until the pavement failed and if the physical state of the pavement did not change during that load history. Multiple strain levels, caused by varying loads and varying physical states, are analyzed by means of Miner's hypothesis.) All investigators have assumed that the ϵ-N relation is linear on a log-log plot. Such a linear relation has been verified repeatedly from laboratory testing (24).

The failure criterion can be established in at least 4 different ways: from theoretical analysis of existing design curves; from an analysis of the performance of in-service pavements, particularly road tests operated under rigid control (box 30); from laboratory fatigue testing (box 40); and from a combination of the procedures given above.

Failure Criterion From Existing Design Curves

Existing design curves, in which the destructive effects of traffic are represented by equivalent axle loads, offer one means for establishing a failure criterion. The design curves are examined to determine one or more pavement sections that are considered adequate for each of several equivalent axle loads. The tensile strains imposed by the standard or base load in those representative structures are then estimated by elastic, multilayered theory. Average environmental conditions in the region for which the design curves are applicable are used to establish representative elastic parameters for the materials in the sections. The computed tensile strains are finally related to the number of load applications to establish the failure criterion. A failure criterion recently proposed by Deen et al. (8, 9) was derived in part from an analysis of existing design curves.

That method for establishing a failure criterion is particularly attractive because of its inherent simplicity and the fact that it results in designs that are fully compatible with an established history of design and performance experience. At the same time, there are several noteworthy drawbacks. First, the failure criterion can be applied with confidence only in geographical areas that have climatic conditions similar to those of the region for which the original design curves were applied. Second, a one-physical-state representation of the structure must be used in the design process. That presents difficulties if the seasonal and daily variations in future traffic volumes are anticipated to deviate significantly from average, region-wide patterns. Third, and most important, that method requires that the original design curves be based solely on the control of failure by fatigue. Most existing design curves consider all distress mechanisms simultaneously. They can be used properly only if conditions can be isolated in which fatigue is the predominant distress mode. Finally, designs that would incorporate new and different types of bituminous mixtures cannot be examined with such a criterion.

Failure Criterion From In-Service Pavements

In-service pavements for which detailed records of traffic and performance have been accumulated offer perhaps the best current base for establishing a failure criterion. Accelerated road tests are particularly useful because of their extensive data base and the controlled nature of those experiments. Accuracy can be improved by laboratory or field testing to determine the elastic parameters of the materials. In addition, analyses can be limited to those test sections known to have failed by fatigue, thereby ensuring that the failure criterion is relevant to the distress mechanism of interest.

If an average annual characterization of the physical state of the pavement is to be used, the analysis proceeds much as before. For a single pavement state, Miner's hypothesis requires that, at failure,

$$\sum_i \frac{n_i}{N_i} = 1 \tag{3}$$

The conventional form of the failure criterion relates N_i to ϵ_i as follows:

$$N_i = k_1 \epsilon_i^{k_2} \tag{4}$$

Combining Eqs. 3 and 4 results in the following equality:

$$\sum_i \frac{n_i}{k_1 \epsilon_i^{k_2}} = 1 \tag{5}$$

For each test section of the road test, n_i is known from the history of load applications, ϵ_i can be computed by elastic, multilayered theory, and the elastic parameters are evaluated under average environmental conditions. (Often only 1 type and magnitude of load is applied to each test section. Equation 5 thus becomes $n = k_1\epsilon^{k_2}$.) An equation similar to Eq. 5 is, therefore, available for each test section. That set of equations can be solved for the unknowns k_1 and k_2 to complete the derivation of the failure criterion.

Improved accuracy can be achieved if due recognition is given to the varying nature of the physical states of the pavement during the test. In this case, Miner's hypothesis at failure is

$$\sum_j \sum_i \frac{n_{ij}}{N_{ij}} = 1 \tag{6}$$

The failure criterion must incorporate the effect of the stiffness of the bituminous mixture on the ϵ-N relation. The generalized form of such a criterion as used by Witczak (39) is

$$N_{ij} = k_1 \epsilon_{ij}^{k_2} E_j^{k_3} \tag{7}$$

in which E_j is the modulus of elasticity of the bituminous mixture when the pavement is in the jth physical state. Constants k_1, k_2, and k_3 can be evaluated by Eqs. 6 and 7 being combined as follows:

$$\sum_j \sum_i \frac{n_{ij}}{k_1 \epsilon_{ij}^{k_2} E_j^{k_3}} = 1 \tag{8}$$

Again n_{ij} is known from the loading history on the test section, and ϵ_{ij} is estimated by elastic, multilayered theory. Evaluation of the resulting set of equations yields suitable estimates of k_1, k_2, and k_3.

The primary disadvantage of a failure criterion so derived is that it is applicable only to the type of bituminous mixture employed in the road test. The failure criterion used by Witczak (39) and developed by Kingham (20) was based on analyses of AASHO Road Test data.

Failure Criterion From Fatigue Testing

A failure criterion can be readily developed from laboratory fatigue testing of the bituminous mixture. Monismith and McLean (27) are among those successfully using such an approach. Test specimens are subjected to repeated flexing until failure, and the ϵ-N relation is determined directly. Laboratory testing is the best way in which the behavior of new and different bituminous mixtures can be accounted for. However, there are several serious obstacles to the use of laboratory-derived failure criteria in routine design. Several of these are noted below.

1. One of the major difficulties is that of defining failure in the laboratory in such a way as to be compatible with failure as defined for the in-service pavement. Brown and Pell (4) suggest that in-service pavement life (applications to failure for a given strain level) is of the order of 20 times the life of a test specimen in the laboratory. Thus, perhaps the best that can currently be achieved with laboratory-derived failure criteria is to obtain an estimate of crack initiation in the in-service pavement. Few techniques are available for quantitatively estimating the progression of cracking in a pavement or for considering varying levels of terminal serviceability.
2. Laboratory fatigue specimens are conventionally subjected to either of 2 types of repetitive loading: controlled-stress or controlled-strain, depending on whether stress (load) or strain (deflection) is controlled during testing. Unfortunately, the number of load applications to failure is extremely dependent on the type of test. It has been hypothesized that in-service pavements are subjected to a type of loading intermediate between those 2 types and that controlled-stress and controlled-strain loadings merely represent end points of an infinite spectrum of possible modes of loading (24). In the absence of suitable means for defining and applying intermediate modes, the problem of which form of laboratory testing to use in design procedures will continue to exist. [Pell (43) gives a discussion of the effect of test type and design recommendations.]
3. Laboratory testing requires selection of a frequency of loading, which is greater than that normally encountered by a pavement in service, and, for pulsating loads, a load duration. Those as well as other laboratory loading variables significantly affect the number of applications to failure. The possibility that rest periods can beneficially alter fatigue response is another variable complicating use of laboratory-derived failure criteria (2, 35).
4. There are certain simplifications in the multilayered, elastic analyses that can cause a departure of predicted from actual pavement response. One of those is that theoretical analysis normally assumes the pavement to have unlimited lateral dimen-

sions and allows no lateral variation in material properties. Barksdale and Hicks (45) discuss an expanded, 2-dimensional, finite-element analysis that can approximate those effects. Complexities in the application of such an analysis probably outweigh any advantages that might accrue by virtue of its use in routine design. Thus, for example, no means are available for readily treating pavement edge support or differential subgrade moisture conditions. Failure criteria derived from in-service pavements would seem intuitively to account for those and other such discrepancies between theory and practicality.

5. Most analyses of highway pavements assume perfect tracking of vehicles; that is, they do not treat the transverse or lateral distribution of vehicle placement. That simplifying assumption may lead to erroneous results if laboratory fatigue criteria are used.

Summary

The most critical component of the fatigue design methodology is establishment of a failure criterion. Unfortunately, no universally acceptable means for establishing such a criterion has been developed. Possibly a combination of methods that uses laboratory testing to establish the behavior of individual mixtures together with analyses of in-service pavements to establish an appropriate definition of failure will evolve as the optimal technique.

At the same time, numerous failure criteria have been established and used by individual investigators. Some of those are shown in Figure 2. The ϵ-N relation is dependent on the modulus of elasticity of the bound materials [the one exception is that used by Brown and Pell (4)], and a smaller number of applications of a given strain level can be tolerated when the mixture has a higher modulus such as during periods of low temperature. Each of the criteria shown in Figure 2 may be adequate in the situation for which it was derived. At the same time, none has universal applicability. They are simply available to be subjected to the test of time as they are applied to the future design of in-service pavements.

Finally, a specific failure criterion can be applied with confidence only if methods used in derivation of that criterion are similar to those used in the design process. That means that methods for characterizing materials, methods for calculating the response of the pavement structure to load, and methods for classifying the physical states of the structure must not be allowed to vary. That caveat is especially important when one uses a failure criterion developed by other investigators. It also casts serious doubts on the practice of using a 1-state pavement representation (average annual conditions) and failure criteria derived from laboratory testing. There is simply no justification for assuming that damage will accumulate under the 1-state simplification in the same manner as it actually accumulates under the varying temperature and moisture conditions of the in-service environment.

Application

The purpose of the analysis phase is to generate estimates of n_{ij} (the number of applications of each traffic load anticipated during the design life while the pavement is in the jth state) and N_{ij} (the number of applications of each traffic load that would cause fatigue failure in the pavement if that load were repetitively applied in the absence of other loads while the pavement is in the jth state). Analyses of existing and projected traffic data yield estimates of n_{ij}. Strains ϵ_{ij} induced in the pavement by the traffic loads are first determined before N_{ij} is estimated. Those strains together with the failure criterion (ϵ-N relation) are used to estimate N_{ij}.

Testing

The analytical routines require, as input, knowledge of the elastic parameters (modulus of elasticity and Poisson's ratio) associated with each material in the pavement structure. In addition, knowledge of the fatigue behavior of the bituminous mixture

Figure 1. Design process.

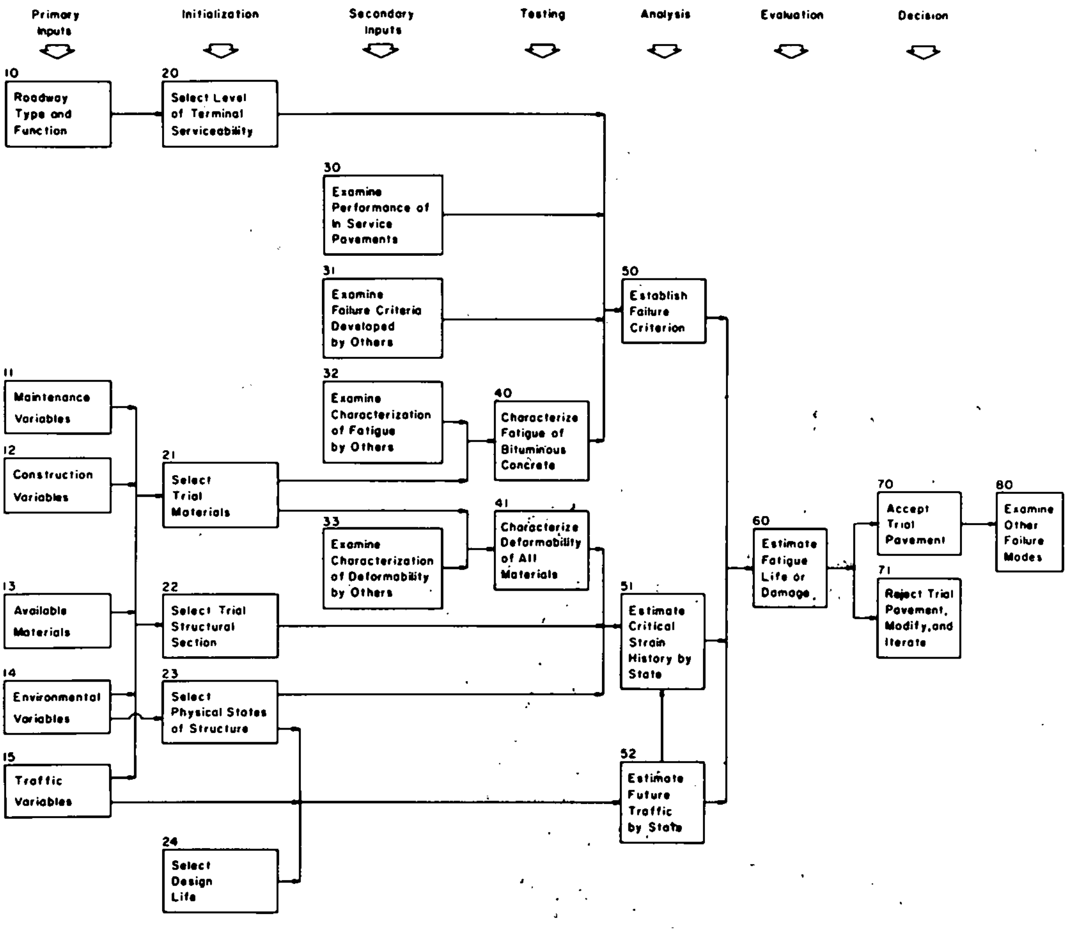
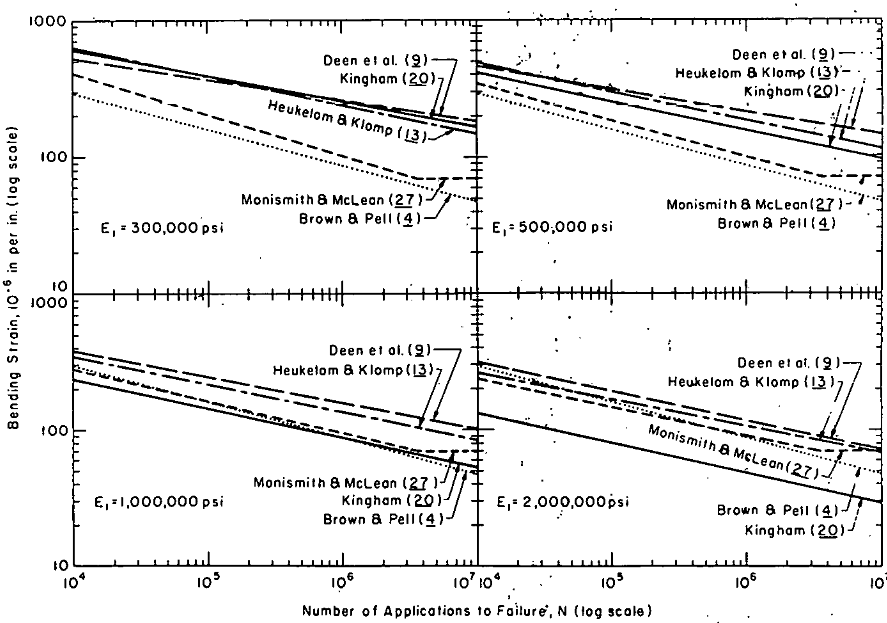


Figure 2. Failure criteria.



under simple laboratory loading conditions may be necessary in the development of a failure criterion. Characterizations of those material responses can, in some instances, be based solely on prior analyses and testing (boxes 32 and 33). [Barksdale and Hicks (45) point out some of the difficulties attendant to the use of deformability characterizations developed by others.] In other instances, such as when new materials are being considered, it may be necessary to evaluate material response by laboratory or field testing.

Detailed procedures for laboratory testing are described elsewhere (43, 45). It is necessary to emphasize, however, that laboratory testing variables, especially those associated with loading, must simulate, to as large a degree as possible, in-service pavement conditions. Nair et al. (28), for example, conclude that asphalt concrete may be characterized as a linear isotropic elastic material for purposes such as fatigue analysis and design. In so doing, however, the elastic parameters must reflect the in-service pavement conditions to adequately account for time of loading, temperature, and stress dependencies.

Physical States

One aspect of the design process shown in Figure 1 deserves additional mention, and that is the selection of the physical states of the structure. The deformability and strength properties of materials within the pavement vary continuously with time in response to environmental factors that alter the moisture and temperature profiles. One approach to that problem is the 1-state representation that uses average, effective, or critical elastic parameters or all of those to analyze structural behavior.

In adopting that approach, Deen et al. (8, 9) found that the average temperature of the bound layers in Kentucky pavements was 64 F. A modulus of elasticity of 480,000 psi, which for the type of mixtures used in Kentucky corresponds with this average temperature, was used in the analytical routines. The subgrade modulus, on the other hand, was a critical modulus evaluated from a soaked laboratory CBR value. That same representation was used both in the development of design relations and in the derivation of a failure criterion largely from an analysis of existing design curves.

Dormon and Metcalf (11) also consider that pavement behavior may be effectively analyzed by means of a 1-state representation. Deformability and strength characterizations of the bituminous layers are based on use of an "effective" temperature, the determination of which requires analysis of the effects on fatigue life of yearly temperature variations for a number of hypothetical pavements. Based on their analysis, a temperature of 50 F was recommended for use in design.

The 1-state representation of in-service pavements is advantageous largely because of its simplicity. No major fault can be found with that procedure as long as the same pavement representation is used both in the analytical routines and in the development of a failure criterion. Difficulties can be anticipated, however, with the use of average parameters if the failure criterion is derived from laboratory fatigue behavior or from evaluations of in-service pavement performance in other geographic regions.

Monismith and McLean (27) have employed a more complex and presumably more accurate representation of in-service pavement conditions by defining 288 physical states. Each state is representative of conditions in a typical hour (for example, 1:00 to 2:00 p.m.) during a given month. The 288 states are probably somewhat optimal for using a laboratory-derived failure criterion. The surface modulus varies for each of the 288 hours as a function of the pavement temperature. Theoretical techniques for estimating pavement temperatures are readily available (23, 27). The subgrade modulus can be varied monthly to reflect seasonal changes in subgrade support due, for example, to frozen conditions in the winter months. Witczak (39) has demonstrated how a varying subgrade modulus can be treated. In addition, seasonal and daily variations in traffic volumes can be most effectively handled.

Determination of how best to categorize the physical states of a pavement is a complex task. Some important considerations that affect the choice include

1. The nature of the failure criterion that is to be used,

2. The availability of techniques for estimating the temperature profile within the bound layers (44) and the relation between modulus and temperature,
3. The availability of techniques for estimating the changes in subgrade modulus with season (44),
4. The nature of the anticipated cyclic variation in traffic volumes, and
5. The desire for simplicity.

Fatigue Distress and Pavement Serviceability

The surface manifestation of fatigue distress is patterned cracking. Such cracking combines with surface manifestations of other distress mechanisms such as thermal cracking, distortion, and disintegration to determine the serviceability of the pavement. Hudson et al. (17) have proposed a technique for defining serviceability whereby weighting functions are used to represent the contribution of each distress manifestation to a reduction in pavement serviceability. If that technique were implemented, pavements could be designed to provide any desired level of terminal serviceability.

To implement such a technique from the standpoint of fatigue distress requires the ability to estimate the extent of fatigue cracking. In theory, that would be a simple task if coupling effects among the various distress modes could be justifiably ignored and if a separate fatigue failure criterion were available for each degree of surface cracking. Unfortunately, few failure criteria are currently available that relate to the extent of surface cracking. McCullough et al. (22) have recently demonstrated an ability to estimate cracking index, a measure of the extent of surface cracking, for 2 AASHO Road Test sections as a function of load history. However, future analyses of in-service pavements, such as the AASHO Road Test, should allow the derivation of suitable failure criteria that incorporate the extent of surface cracking. It seems highly improbable that failure criteria incorporating the extent of surface cracking can be developed from phenomenological analyses that use laboratory test data alone. That is further justification for developing failure criteria on the basis of in-service pavement performance rather than on the singular basis of laboratory test data.

CONCLUDING REMARKS

Well-documented methods, based on a blending of analytical skills and empirical evaluations, are currently available for the design of bituminous pavements to control fatigue distress (41, 42). Like other existing design methods, those methods are not perfect and almost certainly will be improved by future modification. Unlike most existing design methods, however, they have the following capabilities:

1. To treat new types of traffic loading for which little or no historical experience exists,
2. To analyze new types of pavement structures such as full-depth construction,
3. To extrapolate from portions of design curves reasonably well-established by extensive empirical validation to portions for which little or no verification has been possible,
4. To examine new materials to ascertain their effects on pavement performance and to seek ways for better use of existing materials,
5. To provide a rational way for explaining regional differences in pavement performance due to climatic variations and to account for those variations in design, and
6. To enable a proper interpretation of the performance of prototype and road-test pavements.

To fully incorporate all of those extended capabilities into the design process requires a degree of sophistication and complexity uncommon to most current design procedures. However, application of the fatigue-design methods discussed here for routine use need be no more difficult or time-consuming than most current design procedures. Laboratory testing, in excess of that normally undertaken, need not be required. Furthermore, use of the computer need not be required because conventional forms of design charts or nomographs can be and have been developed.

Selected techniques for fatigue-life estimation, on which these design methods are based, are summarized here. Elastic, multilayered analysis provides an acceptable means for estimating critical tensile strains in a pavement structure under realistic traffic loads. Those tensile strains have been found to be the basic cause or determinant of fatigue damage. Accumulation of fatigue damage under varying loads and environmental conditions can be effectively estimated by means of Miner's hypothesis. Proper characterization of the physical states of the pavement structure is essential to the development and application of a fatigue-design methodology and is the primary means of accounting for environmental effects. The burden of computations can be eased substantially by use of repetitive load equivalency concepts and regression and interpolation techniques.

Two particularly crucial aspects of fatigue-life estimation are the development of a suitable failure criterion and the evaluation of elastic material properties. For purposes of routine design, failure criteria derived from an analysis of the performance of in-service pavements would appear to be most suitable at this time. Those criteria reflect the effects of both strain level and modulus of elasticity on the number of load applications causing fatigue failure. Similar methods of materials characterization, physical-state representation, and analysis must be employed both in the development of the failure criterion and in its application to design. Elastic material properties can be evaluated either by laboratory testing or by use of previously published and readily available data.

Capabilities exist for estimating future traffic and environmental variables as input to the design process. However, those capabilities are in need of continual extension and refinement. The impact of maintenance and construction variables on pavement performance has not yet been quantitatively assessed.

REFERENCES

1. Full-Depth Asphalt Pavements for Air Carrier Airports. The Asphalt Institute, College Park, Md., Man. Ser. 11, Dec. 1971.
2. Bazin, P., and Saunier, J. B. Deformability, Fatigue, and Healing Properties of Asphalt Mixes. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1968, pp. 553-569.
3. Bradbury, R. D. Reinforced Concrete Pavements. Wire Reinforcement Inst., 1938.
4. Brown, S. F., and Pell, P. S. A Fundamental Structural Design Procedure for Flexible Pavements. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, Vol. 1, 1972, pp. 369-381.
5. Deacon, J. A. Equivalent Passages of Aircraft With Respect to Fatigue Distress of Flexible Airfield Pavements. Proc. AAPT, Vol. 40, 1971, pp. 194-226.
6. Deacon, J. A. Load Equivalency in Flexible Pavements. Proc., AAPT, Vol. 38, 1969, pp. 465-494.
7. Deacon, J. A., and Monismith, C. L. Laboratory Flexural-Fatigue Testing of Asphalt-Concrete With Emphasis on Compound-Loading Tests. Highway Research Record 158, 1967, pp. 1-31.
8. Deen, R. C., Southgate, H. F., and Havens, J. H. Structural Analysis of Bituminous Concrete Pavements. Kentucky Dept. of Highw., Lexington, Res. Rept. 305, May 1971.
9. Deen, R. C., Southgate, H. F., and Havens, J. H. Structural Analysis of Bituminous Concrete Pavements. Highway Research Record 407, 1972, pp. 22-35.
10. Dormon, G. M. The Extension to Practice of a Fundamental Procedure for the Design of Flexible Pavements. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 785-793.
11. Dormon, G. M., and Metcalf, C. T. Design Curves for Flexible Pavements Based on Layered System Theory. Highway Research Record 71, 1965, pp. 69-84.
12. Edwards, J. M., and Valkering, C. P. Structural Design of Asphalt Pavements for Heavy Aircraft. Shell International Petroleum Co., Ltd., London, 1970.

13. Heukelom, W., and Klomp, A. J. G. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 667-679.
14. Heukelom, W., and Klomp, A. J. G. Road Design and Dynamic Loading. Proc., AAPT, Vol. 33, 1964, pp. 92-125.
15. Heywood, R. B. Designing Against Fatigue. Chapman and Hall, London, 1962.
16. Hicks, R. G., and Finn, F. N. Analysis of Results From the Dynamic Measurements Program on the San Diego Test Road. Proc., AAPT, 1970, Vol. 39, pp. 153-185.
17. Hudson, W. R., McCullough, B. F., and Finn, F. N. Factors Affecting the Performance of Pavement Systems. Proc., ASCE, Vol. 95, No. TE3, Aug. 1969, pp. 505-519.
18. Jones, A. Tables of Stresses in Three-Layer Elastic Systems. HRB Bull. 342, 1962, pp. 176-214.
19. Kasianchuk, D. A. Fatigue Considerations in the Design of Asphalt Concrete Pavements. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, Grad. Rept., 1968.
20. Kingham, R. I. Failure Criteria Developed From AASHO Road Test Data. The Asphalt Institute, College Park, Md., Res. Rept. 71-1, Nov. 1971.
21. Liddle, W. J. Application of AASHO Road Test Results to the Design of Flexible Pavement Structures. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 42-51.
22. McCullough, B. F., Hudson, W. R., and Kher, R. K. Developing an Operational Pavement Design and Management System and Updating It With Elastic Theory. Proc., 3rd Int. Conf. on the Struct. Des. of Asphalt Pavements, London, Vol. 1, 1972, pp. 1061-1073.
23. Monismith, C. L. Some Applications of Theory in the Design of Asphalt Pavements. 5th Annual Nevada St. and Highw. Conf., Univ. of Nevada, Reno, March 1970.
24. Monismith, C. L., and Deacon, J. A. Fatigue of Asphalt Paving Mixtures. Proc., ASCE, Vol. 95, No. TE2, May 1969, pp. 317-346.
25. Monismith, C. L., and Epps, J. A. Asphalt Mixture Behavior in Repeated Flexure. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, Rept. TE-69-6, Dec. 1969.
26. Monismith, C. L., Kasianchuk, D. A., and Epps, J. A. Asphalt Mixture Behavior in Repeated Flexure: A Study of an In-Service Pavement Near Morro Bay, California. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, Rept. TE-67-4, 1967.
27. Monismith, C. L., and McLean, D. B. Structural Design Considerations. Annual Meet. of AAPT, Cleveland, Feb. 1972.
28. Nair, K., Smith, W. S., and Chang, C. Y. Characterization of Asphalt Concrete and Cement-Treated Granular Base Course. Materials Research and Development, Inc., Oakland, Calif., Feb. 1972.
29. Painter, L. J. Analysis of AASHO Road Test Data by the Asphalt Institute. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 84-97.
30. Peattie, K. R. A Fundamental Approach to the Design of Flexible Pavements. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 403-411.
31. Peattie, K. R. Stress and Strain Factors for Three-Layered Elastic Systems. HRB Bull. 342, 1962, pp. 215-253.
32. Pell, P. S. Fatigue Characteristics of Bitumen and Bituminous Mixes. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1963, pp. 310-323.
33. Peutz, M. G. F., van Kempen, H. P. M., and Jones, A. Layered Systems Under Normal Surface Loads. Highway Research Record 228, 1968, pp. 34-45.
34. Thickness Design for Concrete Pavements. Portland Cement Assn., Chicago, 1966.
35. Raithby, K. D., and Sterling, A. B. The Effect of Rest Periods on the Fatigue Performance of a Hot Rolled Asphalt Under Reversed-Axial Loading. Proc., AAPT, Vol. 39, 1970, pp. 134-152.
36. 1963 Design Charts for Flexible Pavements. Shell International Petroleum Co., Ltd., London, 1963.

37. Shook, J. F., and Finn, F. N. Thickness Design Relationships for Asphalt Pavements. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1962, pp. 52-83.
38. Warren, H., and Dieckman, W. L. Numerical Computation of Stresses and Strains in a Multiple-Layer Asphalt Pavement System. Chevron Research Corp., unpubl. rept., 1963.
39. Witczak, M. W. Design of Full-Depth Asphalt Airfield Pavements. The Asphalt Institute, College Park, Md., Res. Rept. 71-2, Nov. 1971.
40. Terrel, R. L. Examples of Approach and Field Evaluation: Research Applications. Paper in this Special Report.
41. Witczak, M. W. Fatigue Subsystem Solution for Asphalt Concrete Airfield Pavements. Paper in this Special Report.
42. Havens, J. H., Deen, R. C., and Southgate, H. F. Pavement Design Schema. Paper in this Special Report.
43. Pell, P. S. Characterization of Fatigue Behavior. Paper in this Special Report.
44. Hudson, W. R. Other Input Variables: Traffic and Environmental. Paper in this Special Report.
45. Barksdale, R. D., and Hicks, R. G. Material Characterization and Layered Theory for Use in Fatigue Analysis. Paper in this Special Report.

The basic premise of this symposium is the assumption that fatigue cracking in an asphalt pavement is "bad" and that such cracking reduces the long-term performance capability of the pavement. Probably without exception, engineers, particularly maintenance engineers, would agree with that conclusion. However, how much cracking is bad, what kinds of cracking are bad, and what happens to a pavement after cracking occurs are all questions that need to be answered before cracking predictions are undertaken. This paper attempts to relate fatigue cracking to performance. It is hypothesized that one of the objectives of any structural design system is to provide a pavement that will resist fatigue cracking to such an extent that premature maintenance will not be required.

Relation Between Cracking and Performance

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In December 1970, the Highway Research Board conducted a special workshop dealing with the structural design of asphalt concrete pavements. The primary goal of the workshop was to review the current status of structural design and to evaluate and assimilate ways of applying research findings to future design procedures. Emphasis was placed on the need for relating distress to performance and performance in turn to the present serviceability index (riding quality) of a pavement.

One of the workshop groups was charged with responsibility for discussing distress and pavement performance. The report (1) from that group indicated unanimous agreement that a present serviceability rating or present serviceability index evaluation system is the most satisfactory method currently available for evaluating pavement performance.

However, it would appear that cracking alone does not correlate well with riding quality as quantified by the present serviceability index (PSI) equation developed by the AASHO Road Test staff (2). For example,

$$\mathrm{PSI} = 5.03 - 1.91 \log (1 + sv) - 1.38\, \overline{RD}^2 - 0.01\, (C + P)^{1/2}$$

where

sv = slope variance, a measure of longitudinal roughness,
$\overline{RD}$ = average rut depth, and
$C + P$ = area of class 2 + class 3 cracking plus patching, per 1,000 ft^2.

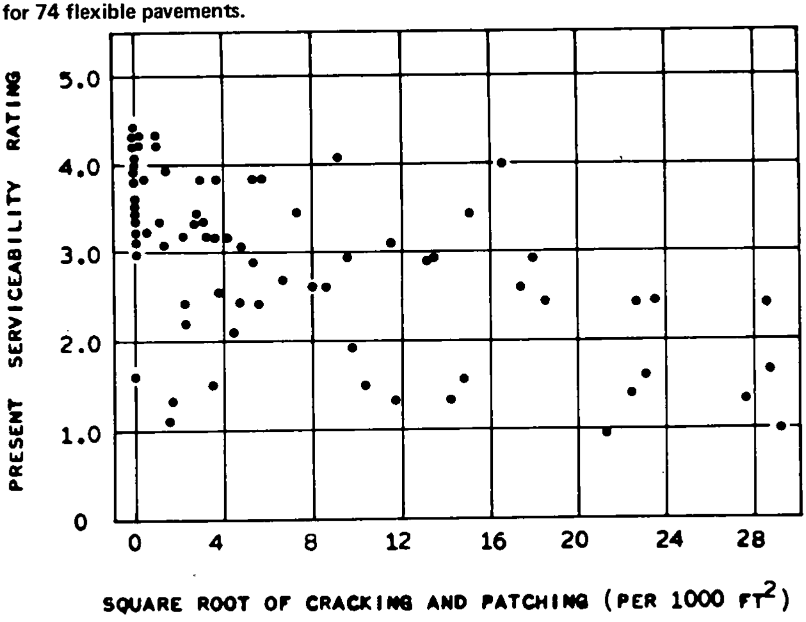
A numerical evaluation of that relation will indicate that when there is 100 percent cracking, that is, 1,000 ft^2 of cracking per 1,000 ft^2 of pavement, the serviceability is reduced by 0.33. The suggestion here is that cracking may not influence riding quality in a particularly significant amount. Longitudinal roughness, which would logically be expected to be the most significant single parameter in the evaluation of riding quality, has simply overwhelmed other factors such as rut depth and cracking plus patching. It was suggested at the HRB workshop that a high correlation exists between the present serviceability rating and the amount of cracking and patching on a given pavement. Specifically, it was indicated that present serviceability ratings obtained by the use of only cracking and patching would give a correlation coefficient of about 0.8 and those obtained by the use of roughness would give a correlation coefficient of 0.9. However, the AASHO Road Test report (2) does not appear, at least in this particular example, to give that reliable a coefficient of correlation. Figure 1 (2) does suggest that, when the amount of cracking exceeds approximately 40 percent (400 ft^2 per 1,000 ft^2) of the total area, none of the pavements would have an acceptable riding quality of 2.5.

In all probability, the subjective opinions of most engineers would be that 40 percent of class 2 or class 3 structural cracking in an asphalt pavement would be highly excessive and totally unacceptable in terms of the adequacy of a particular pavement structure. It is pertinent to note that the PSI includes cracking plus patching. The specific area of cracking included in the equation cannot be distinguished from patching. Hence, the areas of cracking shown in Figure 1 could be relatively small, and good quality patches would not contribute to increased roughness. Thus, efforts to analyze the PSI equation could be misleading.

In 1963, Rogers et al. (3) reported results of a nationwide survey of pavements that were scheduled to be rehabilitated. Efforts to correlate PSI with the amount of class 2 and class 3 cracking plus patching indicate that little, if any, correlation exists between those 2 factors. Examination of the data included in that report is limited because areas are reported as the sum of cracking and patching. However, the data indicate that (a) for pavements with less than 5 percent cracking plus patching, the PSI was above 2.0; and (b) for pavements with more than 45 percent cracking plus patching, the PSI was below 2.5. A limiting amount of cracking and patching would be between 5 and 45 percent of the pavement area. Admittedly, that is a very loose association, but, when added to other bits of information, it may be useful.

Those at the HRB workshop were unable to postulate a specific method by which distress could be related to performance. They were in agreement that the occurrence of cracking can be damaging to the continued performance of the pavement. They also agreed that when cracking occurs, water will enter through the pavement into the base

Figure 1. Present serviceability rating versus square root cracking and patching for 74 flexible pavements.



and subgrade and cause increased roughness because of heaving or consolidation. Pell (10) indicates that a considerable amount of crack propagation will have to occur over a wide area before any serious deterioration of the pavement structure results from the possible penetration of water or from the increase in stresses in the underlying layers or from both of these.

On the other hand, Terrel (9) indicates,

> Fatigue cracks were frequently developed during the summer and autumn months when test pavements received the first phase of loading. Provided there was no accompanying change in subgrade or base courses, the number of load repetitions supported were surprisingly large. . . . Intrusion of water through the cracked surface and the usual spring thaw tended to weaken the underlying material. When testing was resumed in the spring, total failure quickly resulted. That was a result not of further fatigue distress but of subgrade failure.

Apparently, Terrel found that with cracking local failure will be accelerated by water. Pell considers that this will not necessarily propagate areally and, therefore, could be controlled or would not significantly influence the total pavement.

During the AASHO Road Test, the occurrence of class 2 cracking was observed at approximately 50 percent of the traffic volume necessary to reduce the riding quality to a level of 2.5 (2). Clearly then, it can be expected that a certain amount of cracking is acceptable in terms of riding quality but not acceptable in terms of structural integrity. On the Zaca-Wigmore Test Road in California, maintenance engineers established the terminal amount of cracking to be 10 percent. That is, after the pavements exhibited 10 percent cracking based on the total area, it was determined that maintenance was necessary in order to hold that pavement together and to reduce the eventual need for expensive rehabilitation.

Examination of some pavement condition survey forms may be useful in attempting to equate cracking to performance. In Washington (3), a system is used of negative values for various types of structural deficiencies. The negative values for alligator cracking are as follows:

Percentage of Area/Station	Hairline	Spalling	Spalling Plus Pumping
1 to 24	2	5	10
24 to 49	5	10	15
50 to 74	10	15	20
75+	15	20	25

It is perhaps significant that the first percentage division is set at 24 percent. Riding quality is combined with distress to summarize pavement ratings on a scale of 0 to 100. The reduction in overall rating associated with 1 to 24 percent cracking would be comparable to one-half point on the PSI scale of AASHO. That seems to suggest fairly significant sensitivity to cracking by that method.

Another evaluation of the Washington method is to compare the total amount of reduction by cracking to the total allowable deduction for various classes of flexible pavements. For example, the limiting rating for Interstate and principal pavements has been set at 60. Assume that the riding quality is good (equivalent to a PSI of 4.0). With no cracking, the rating would be 84; with 75 percent of the area exhibiting cracking, the rating would be 72. Assume that riding quality is fair (equivalent to a PSI of 2.5). With 75 percent cracking, the rating would be 62. It appears that, if only riding quality and cracking are considered, extensive cracking would be permitted and still be acceptable. In all probability, that is a somewhat questionable conclusion because other forms of distress (e.g., rutting) have probably occurred in practice to cause the pavement rating to fall below 60 long before cracking reached 75 percent of the area.

Forbes (5) has suggested a similar classification for cracking except that the first separation on the amount of cracking is 15 percent. Azarnia (6) in evaluating maintenance criteria for county roads indicates that alligator cracking at less than 5 percent of the area should not require maintenance; however, maintenance could be required for 15 percent cracking depending on the level of severity. Hughes (7) has also reported on some of the problems related to structural deficiencies. Although this report was issued in 1971, several of the comments are still very pertinent.

> 1. The subject of pavement deficiencies has received the most attention since the initiation of the structural rating system. There are, of course, obvious deficiencies which affect pavement performance and develop a need for resurfacing. The main difference of opinion among engineers is the relative seriousness of these deficiencies.
> 2. The first step in the development of the rating system, then, was to hold discussions with individuals knowledgeable about pavement deterioration to enumerate and define the deficiencies which indicate the structural condition of a roadway. In addition, the relative seriousness of the deficiencies were discussed and numerical factors indicating the same were assigned.

The condition rating in Minnesota is obtained by averaging the PSR (riding quality) and the structural rating, each on a scale of 0 to 5. A weighting factor is applied to the various forms of structural distress, and information is corrected to a per-mile basis. By the suggested procedures, it would require almost 30 percent of the area to have alligator cracking before the pavement would be unacceptable. That is, if the riding quality is 2.5 or better, the area of cracking would need to be about 30 percent before the overall rating would drop below 2.5.

On the basis of the discussion given above, it is possible to draw the following conclusions:

1. Fatigue cracking in asphalt pavements is damaging to the continued successful performance of that pavement;
2. The initial occurrence of cracking may not be so crucial to the overall performance as some acceptable level generally indicative of a pending period of accelerated deterioration; and
3. The acceptable level of fatigue cracking could be between 10 and 30 percent depending on the amount of other types of structural distress and the riding quality.

Thus, to be helpful to the highway engineer, the output variable of cracking as predicted from research should include not only some estimate of initial cracking but also the rate of progression of cracking with time. McCullough et al. (8) indicated that it may be possible to predict the accumulation of cracking with time.

There are difficulties in using cracking as an output variable for field projects. That was probably best demonstrated on the WASHO Road Test in Idaho. On that project, cracking was considered the dependent variable or the performance rating factor. Design requirements were essentially based on the thinnest test sections considered not to have been damaged by the test traffic; in other words, those sections that basically had 0 percent distress or cracking. Examination of graphical representations of that information suggests that, if as little as 5 percent cracking had been tolerated, the allowable thickness requirements could have been reduced by approximately 3 in. for those sections surfaced with 2 in. of asphalt concrete and about 1 in. for those sections surfaced with 4 in. of asphalt concrete.

A second somewhat confounding aspect of the WASHO and AASHO data is that there were long periods at certain times of the year in which no cracking occurred in the pavement. Another problem is the ability to associate the various types (forms) of cracking with structural distress.

In summary, it would appear that the need to predict cracking is important and that most highway engineers agree that cracking is the first indication of the loss of load-carrying capacity of a given pavement construction. The initial occurrence of cracking is not so crucial to overall performance as the need to be able to predict some rate of propagation of cracking to some limiting amount.

At the present time, there does not appear to be any method of objectively quantifying cracking as a performance parameter. Performance data from the AASHO Road Test suggest that some cracking can be tolerated without serious loss in riding quality. Several papers in this Special Report and condition survey procedures indicate that some cracking is permissible. However, an upper limit needs to be established as a design criterion, and the further development of fatigue research needs to concentrate on some method for estimating the areal propagation of cracking as suggested by McCullough.

The only reasonable way to quickly determine the limiting amount of cracking is to ask the engineers who are responsible for the maintenance. Combining subjective evaluations of the future utility of a given pavement with objective measurements of the amount of cracking existing on the pavement with a well-planned performance feedback system should make it possible to develop useful cracking criteria. In this way, agencies can begin to establish a quantitative association between cracking and performance in terms of the need for specific maintenance strategies necessary to protect the basic investment.

REFERENCES

1. Structural Design of Asphalt Concrete Pavement Systems. HRB Spec. Rept. 126, 1971.
2. The AASHO Road Test: Report 5—Pavement Research. HRB Spec. Rept. 61E, 1962.
3. Rogers, C. F., Cashell, H. D., and Irick, P. E. Nationwide Survey of Pavement Terminal Serviceability. Highway Research Record 42, 1963, p. 26.
4. LeClerc, R. V., and Marshall, T. R. A Pavement Condition Rating System and Its Use. Proc., AAPT, 1969, pp. 280-295.
5. Forbes, C. E., and Currier, D. E. Surface Evaluation for Budgeting and Planning. 49th Annual Meeting of WASHO, 1970.
6. Azarnia, G. Development of Performance and Maintenance Criteria for County Roads. Univ. of Washington, Master's thesis, 1972.
7. Hughes, P. C. Development of a Rating System to Determine the Need for Resurfacing Pavements. Off. of Res. Coord., Minnesota Dept. of Highways, Invest. 189, 1971.
8. McCullough, B. F., Hudson, W. R., and Kher, R. K. Developing an Operational Pavement Design and Management System and Updating With Elastic Theory. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, 1972, p. 1061.
9. Terrel, R. L. Examples of Approach and Field Evaluation: Research Applications. Paper in this Special Report.
10. Pell, P. S. Characterization of Fatigue Behavior. Paper in this Special Report.

Although considerable work has been completed on laboratory testing of asphalt concrete to determine fatigue properties, more work must be done on predicting fatigue behavior in the field. The initiation and progression of fatigue cracks in a pavement normally take on the familiar "alligator" or "chicken-wire" shape. However, their point of initiation within the pavement structure or within the life-span of the pavement is not well understood. This paper describes several projects that were developed as pavement experiments to investigate several aspects of performance. The projects selected are representative in terms of both simple field observation of fatigue cracking and complete studies that include data on materials characterization, loading history, and other aspects.

Examples of Approach and Field Evaluation: Research Applications

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Five separate road tests or experiments have been selected by the author in an attempt to illustrate 2 factors: (a) the initiation and development of fatigue or fatigue-related cracks and (b) the prediction of fatigue cracking as part of controlled experiments where knowledge of the pavements was sufficient to provide input for modeling their behavior. The Brampton and San Diego projects are included in the first category, and the Folsom and Morro Bay projects are in the second. Tests at the Washington State University test track provided an opportunity for both observation and prediction.

BRAMPTON TEST ROAD

During the summer of 1965, a full-scale test road was constructed near Brampton, Ontario (1). Although many pavement sections were included in that experiment, two were selected here to illustrate fatigue-related crack development.

The first contained 3½ in. of asphalt concrete, 2 in. of crushed-stone base, and 6 in. of sandy-gravel subbase. After an initial period of service, short transverse cracks began to develop in the outer wheelpath as shown in Figure 1. Progression to more advanced forms of alligator cracking (Figs. 2 and 3) occurred rather rapidly and is somewhat typical for that type of failure. The load-associated cracking shown in those figures is perhaps closest to the pure fatigue failure and has been observed by the author to be frequently associated with asphalt pavements overlying unbound granular bases.

The second example at Brampton illustrates that crack initiation may be non-load-associated in many instances but may eventually develop into fatigue cracking. Low-temperature cracking, as shown in Figure 4, or reflection cracking from underlying cement-bound layers, as shown in Figure 5, often occurs soon after construction. Those breaks in the pavement surface may permit initially higher than normal deflections because of free boundary conditions or because of influx of water into underlying layers or both.

How non-load-associated cracks rapidly develop into load-associated cracking is shown in Figures 5, 6, and 7. The surface crack occurred about 2 to 3 months after construction and was reflected from the underlying cement-treated base. Alligator cracks began to appear around that initial crack (Fig. 5), and additional secondary cracking in the wheelpath (Fig. 6) was further evidence of load association. In many instances, those pavements with cement-treated bases were also badly rutted as shown in Figure 7. That observation and the associated high deflection measurements suggested that the cement-bound material was breaking down and reverting to a situation analogous to a granular base. As a result, failure was markedly accelerated.

SAN DIEGO TEST ROAD

Another example of an extensive test project in which observations are still under way is the experiment on Sweetwater Road near San Diego, California (2). Constructed in the mid-1960's, the project was a cooperative study of 8 base types with 4 levels of thickness. After 4 years of in-service operation, none of the sections showed any significant distortion or permanent deformation. However, although no photographs were available, many of the pavement sections showed considerable load-associated cracking (2). Based on observations by Finn (3), about 27 percent of the test sections exhibited cracking of that type. In several instances, the cracking may have been associated with longitudinal cracks that occurred along the outer edge of the pavement and that were not associated with traffic. However, many of the sections were of full-depth design, and failure appeared to be somewhat different from that observed at the Brampton test road.

FOLSOM TEST PROJECT

Several full-scale projects in California have been reported by Monismith et al. (4), in which considerable testing and analysis were used in an attempt to predict fatigue life. The Folsom project, constructed by the California Division of Highways between Sacramento and Placerville, had a pavement cross section as shown in Figure 8. The subgrade and untreated materials in the pavement were tested in repeated load triaxial compression to obtain the resilient moduli. The asphalt concrete was tested for fatigue properties.

Two variations of the subgrade were considered; those were part of a compaction study. A range of moisture for a particular stress condition was used so that a relation such as that shown in Figure 9 could be used to represent the subgrade material under various construction, load, and environmental conditions. In that figure, isoclines of resilient modulus were developed over a range of moisture-density conditions.

The rounded-gravel subbase material was used to fabricate cylindrical specimens for repeated load tests over a range of cell pressures. For those tests, both axial and circumferential deformations were measured, so that resilient modulus and Poisson's ratio were determined as follows:

$$M_R = 7{,}730\ (\theta)^{0.46}$$

$$\nu = 0.13 + 0.05\ (\sigma_1/\sigma_3)$$

where

θ = sum of principal stresses,
σ_1 = vertical repeated stress, and
σ_3 = lateral cell pressure

A crushed-gravel base-course material was tested in the same manner as was material for the subbase, and modulus and Poisson's ratio relations were determined as follows:

$$M_R = 3{,}470\ (\theta)^{0.65}$$

$$\nu = 0.16 + 0.08\ (\sigma_1/\sigma_3)$$

Figure 1. Initial transverse cracks in outer wheelpath of Brampton test road.

Figure 2. Beginning of alligator fatigue cracks.

Figure 3. Complete alligator cracking prior to repair.

Figure 4. Non-load-associated crack that was thermally induced and may act as catalyst to fatigue cracking.

Figure 5. Initial crack that was reflected up from cement-treated base and is starting point for fatigue cracking.

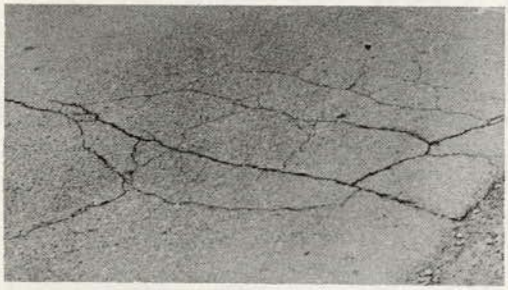


Figure 6. Cracking shown in Figure 5 that has developed to complete alligator cracking in wheelpath.

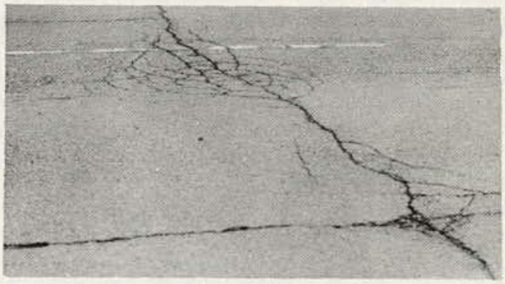


Figure 7. Section with cement-treated base showing considerable pavement deformation prior to cracking.

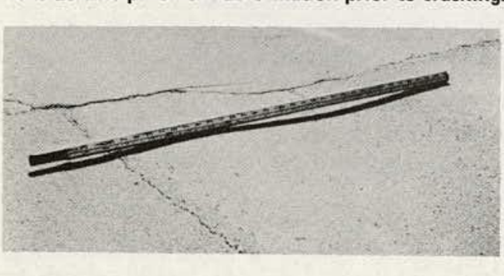


Figure 8. Structural pavement section of Folsom project.

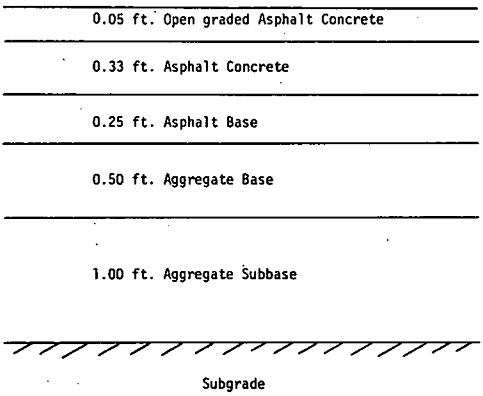
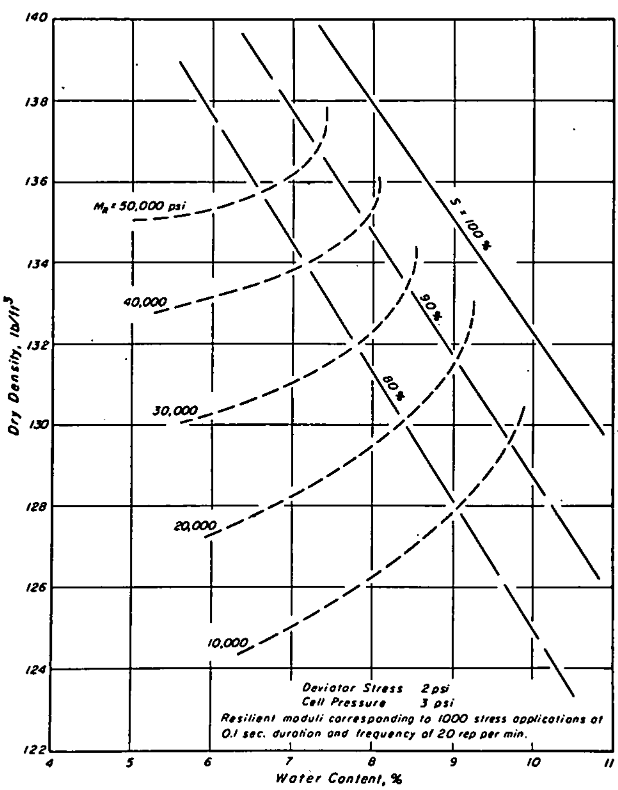


Figure 9. Water content-dry density-resilient modulus relation for subgrade soil.



The asphalt-concrete surface course was tested for flexural stiffness characteristics by the use of beam specimens over a range of temperatures. Stiffness values for 2 ranges of loading times are shown in Figure 10. For purposes of pavement response, Poisson's ratio was assumed to vary from 0.30 to 0.35.

Figure 11 shows the Folsom pavement section as well as the material characteristics for each layer. Once the pavement was characterized by the laboratory tests, it was desirable to determine how measured surface deflections compared with those calculated by the CHEV 5L program. Figure 12 shows a comparison of those deflections and also the effect of asphalt concrete stiffness and subgrade moisture (inasmuch as those 2 variables had the most influence on deflection).

In addition to stiffness modulus of the asphalt-concrete surface course, fatigue characteristics were also determined. Results of both controlled-stress and controlled-strain tests at 68 F are shown in Figures 13 and 14 for the control section. Similar relations were determined for the other test sections but are not shown.

Estimating the service life of the pavement involves systematically characterizing the materials, traffic, and environmental factors and then comparing the predicted life and cracking with that actually observed. The steps followed in that estimation are summarized below.

1. Estimate the truck traffic distribution throughout the day in hourly increments for the various axle-load groups based on several assumptions. All of the traffic variables were related.

$$AHT_{ij} = ADTT \cdot \frac{A_j}{100} \cdot \frac{HT_i}{100}$$

where

$ADTT$ = average daily truck traffic, one direction in design lane;
A_j = percentage of truck traffic of class j (2 axle, 3 axle, and so on);
HT_i = percentage of truck traffic in the hourly interval i; and
AHT_{ij} = number of operations of class j in hour i (daily).

$$AXLD_{ik} = \sum_{j=1}^{S} (AHT_{ij} \cdot WLF_{kj})$$

where

WLF_{kj} = wheel load factors to relate axle class j to axle-load group k, and
$AXLD_{ik}$ = matrix of the number of axle loads of group k in each hour i (on a monthly basis).

The $AXLD_{ik}$ values were expanded to an annual basis by incorporating climatic information as described in step 2.

2. Estimate the stiffness moduli for the asphalt-concrete layer based on climatic data for the Folsom area. Those data were used in a computer program that was modified to provide mean stiffness and temperature at the beginning and end of each hourly increment throughout the day. That approach allowed the use of a single modulus value for the full depth of asphalt concrete and simplified the analysis.

3. Compute stress and deformation by the use of the multilayer linear elastic computer solution, and estimate the bending strains on the bottom side of the asphalt bound layers. The range of those values is shown in Figure 15.

4. Estimate the fatigue performance of asphalt concrete from the road section based on data similar to that shown in Figures 13 and 14. For pavements with asphalt sections of an intermediate thickness, a mode factor (i.e., stress- or strain-controlled) should be considered. A range of mode factors that are intermediate to the limits obtained can be determined in laboratory fatigue testing depending on whether the tests are stress-controlled or strain-controlled. A comparison of mode factors is shown in Figure 16 and can be used for variable thickness pavements.

Figure 10. Computed relation between mixture stiffness and temperature.

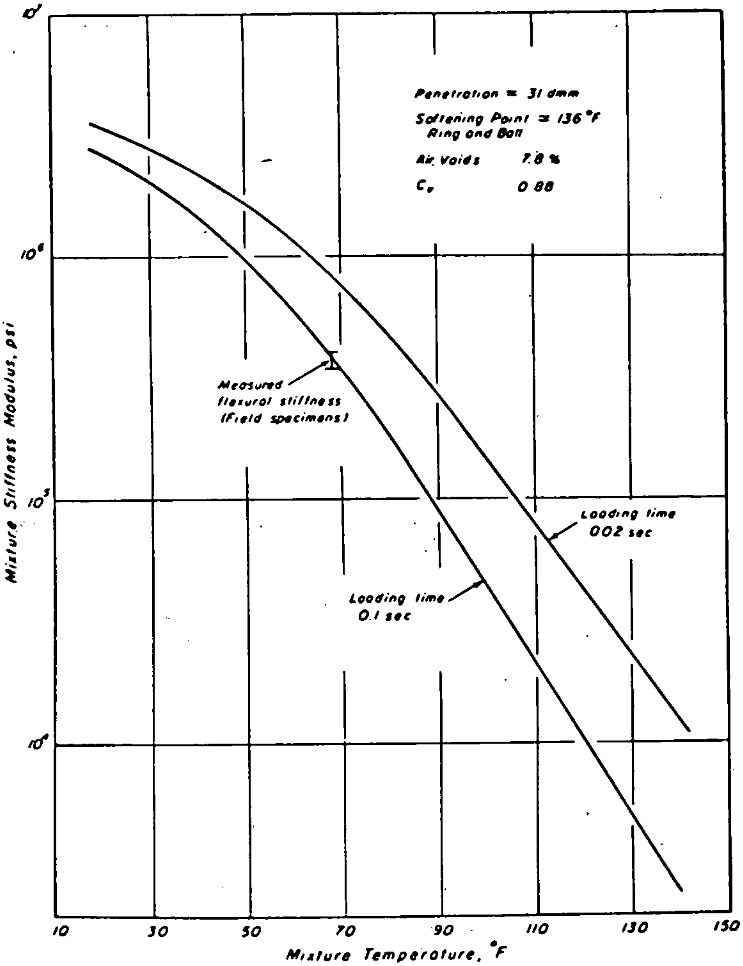


Figure 11. Pavement section used in analysis.

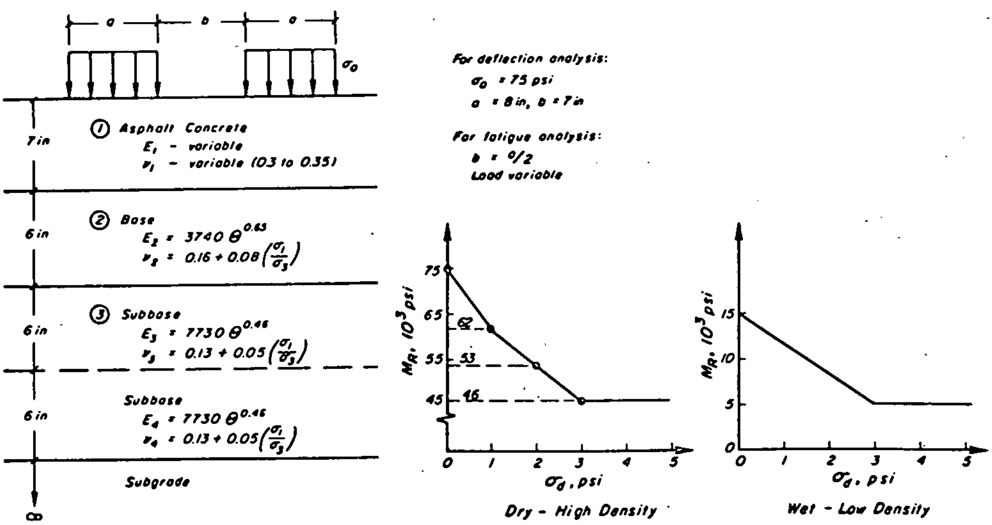


Figure 12. Computed and observed pavement deflections.

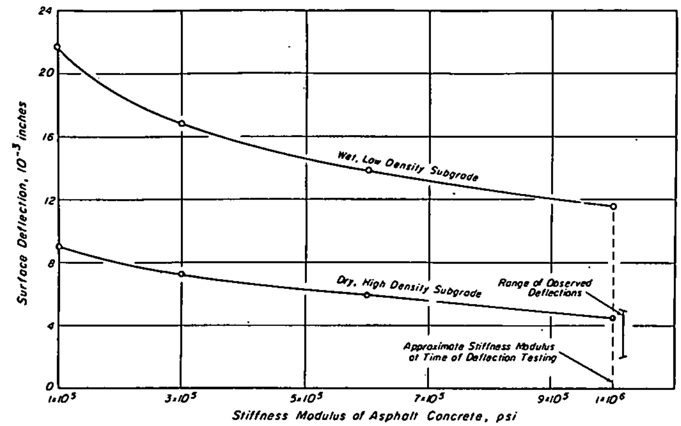


Figure 13. Initial mixture bending strain and applications to failure.

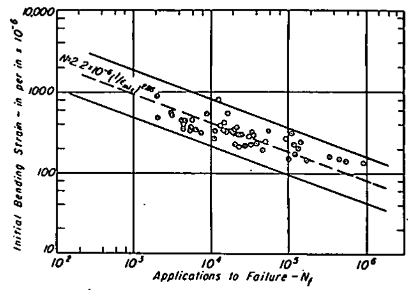


Figure 14. Mixture bending stress and applications to failure.

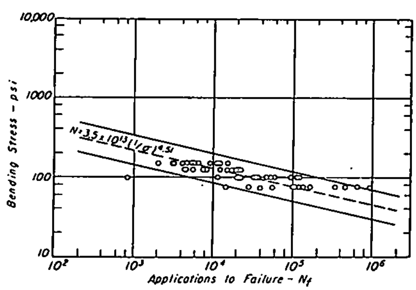


Figure 15. Bending strain on underside of asphalt layer and axle load for stiffness range.

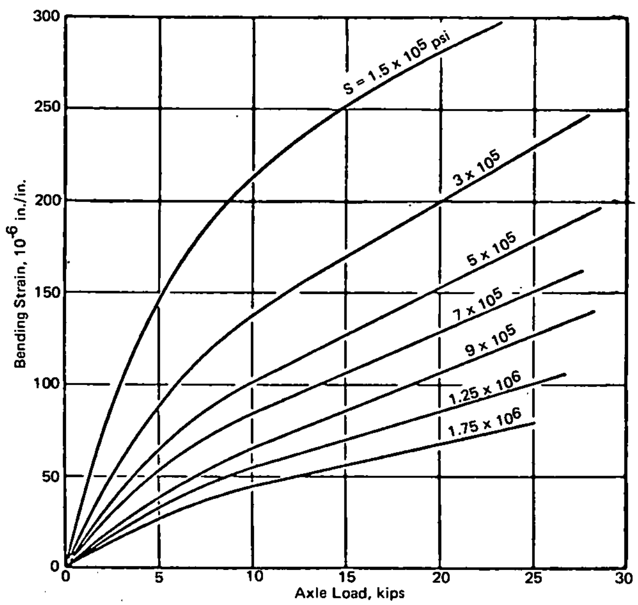
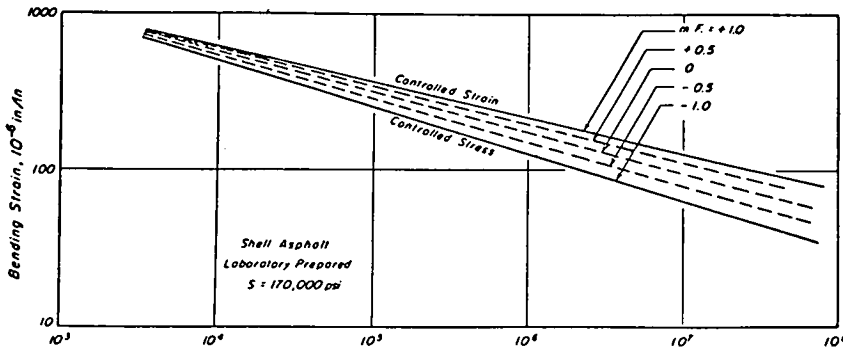


Figure 16. Fatigue curves for series of modes of loading.



5. Predict fatigue life by assembling all the required data so that, for each increment of time, a condition representing the pavement in terms of strain, mode factor, and stiffness is easily accessible. The number of repetitions to failure were designed $ENNF_{\ell k}$, where ℓ is the stiffness group and k is the axle-load group. Then,

$$\text{Annual damage} = \sum_{\ell} \sum_{k} (APL_{\ell k}/ENNF_{\ell k})$$

Traffic values were then incremented by an expansion factor (3 percent), and the process was repeated until the total damage was equal to one.

Fatigue life predictions were made by the use of 3 sets of strain-repetitions to failure relations. The relations and the predicted fatigue life are given below (4).

1. Fatigue life was based on tests of laboratory specimens prepared with the control asphalt. Individual estimated lines were linearly extrapolated to the full range of strain computed. Those data resulted in a predicted fatigue life of 9.5 years.
2. The data were the same as those used in step 1 except that the assumption was made that strains less than 70×10^{-6} in./in. caused no fatigue damage (i.e., endurance limit). The data resulted in a predicted fatigue life of 12 years.
3. A set of relations was estimated from results of tests on specimens obtained from the field surface-course layer. Strains less than 70×10^{-6} in./in. were again excluded. Those data resulted in a predicted fatigue life of 6 years.

The pavement in the Folsom project was designed for a 10-year life by the California Division of Highways using conventional techniques. Based on appropriate traffic data for 1970, suitable extrapolations, and other tests and analysis, the pavement was predicted to perform well on the basis of fatigue life. In other words, a predicted life of 9 to 12 years based on fatigue data appears to be reasonable.

MORRO BAY PROJECT

Other projects, similar to the Folsom project, were a part of the total program described by Monismith et al. (4). One of those near Morro Bay, California, was analyzed by similar techniques but was based on much less sophisticated fatigue data. One conclusion was that a single fatigue curve may not be sufficient for prediction of fatigue life when the total input is so complex.

The Folsom project was newly constructed so that sufficient time had not elapsed for actual fracture to initiate in the field sections by the time of reporting. The Morro Bay project was constructed in 1963, however, and offered an opportunity to observe field cracking. The total analysis, similar to that described above, indicated that some distress might show up in 1965. Field sampling did not show any such evidence, but resampling in 1967 revealed the apparent initiation of fatigue cracking. Figure 17 shows the pattern of observed cracks on the bottom side of several slabs of asphalt concrete removed from the pavement.

WASHINGTON STATE UNIVERSITY TEST TRACK

The special testing facility at Washington State University has provided another opportunity to study fatigue behavior in the field. That apparatus, shown in Figure 18, permits a careful control of the traffic factors, which were difficult to estimate for the in-service pavements discussed in the previous section. Details of the facility have been described elsewhere (5, 6).

Although 4 circular test rings have been completed at the track, the test series conducted from October 1968 to August 1969 was selected for analysis here. That particular test ring consisted of 12 pavement sections (Table 1). The main experiment was the investigation of the 3 main base types, all covered by a uniform 3-in. asphalt concrete surface. Loads were applied to the pavement sections by the special 3-armed loading frame at speeds varying from 5 to 20 mph. Dual tires had a total load of 10,600 lb, and a special control mechanism caused the loads to move slowly from one side to the other while all 3 sets of tires were simultaneously revolving around the track.

After the construction of the Washington test track, loading was initiated in and continued through the autumn of 1968 when weather was extremely wet but was suspended during freezing conditions throughout the winter. When testing was resumed the following March, very little additional rainfall occurred, and the temperatures were steadily increasing until the conclusion of the test in August 1969.

Performance as measured by Krukar and Cook was in 2 phases: (a) the number of load applications until the appearance of the first cracking, and (b) the applications until the section was completely destroyed and the test was stopped for repairs. In most of the pavement sections tested, surface cracks first appeared as short transverse cracks very similar to those at the Brampton test road. Figure 19 shows an example of those cracks that have been accentuated with chalk. That form of cracking was typical for pavements with untreated base courses and with thinner base sections of emulsion and asphalt cement treatment.

Figure 20 shows an example of the next phase of crack advancement. In addition to further cracks, an appreciable amount of permanent deformation has occurred in the wheelpath. That type of failure was often of a local nature and may have been associated with weak or uncompacted underlying materials. A more advanced form of alligator cracking is shown in Figure 21 and is somewhat typical for most of the thinner sections. Fatigue cracks were frequently developed during the summer and autumn months when those test pavements received the first phase of loading. Provided there was no accompanying deformation in subgrade or base courses, the number of load repetitions supported was surprisingly large. In other words, once the primary fatigue cracks appeared, no further development of failure appeared during many thousands of repetitions (note in Fig. 21, 588,000 repetitions).

During the wet fall, rainwater accumulated in the depressed areas and thoroughly soaked the untreated bases and subgrade. Intrusion of water through the cracked surface and the usual spring thaw tended to weaken the underlying material. When testing was resumed in the spring, total failure quickly resulted as shown in Figure 22. That was a result not of further fatigue distress but of subgrade failure.

Several of the thicker (up to 9.5 in.) all-asphalt-concrete and asphalt-treated base sections served considerably longer before distress began to appear. Eventually, when the typical short transverse cracks appeared they were often accompanied by evidence of pumping. Figure 23 shows small amounts of the silt subgrade being exuded at the surface. Other thick sections did not exhibit surface cracks. However, when the pavements were removed and examined, there was considerable evidence of alligator cracking on the underside of the asphalt layers as shown in Figure 24. Subgrade soil had also been pumped into that crack system. The fact that cracks were well developed at the bottom and practically nonexistent at the surface was further evidence that the failure was of the fatigue type and that cracks initiate at the bottom and propagate toward the surface.

It appears feasible to predict the advent of fatigue cracking when sufficient information and test data are available. A brief attempt at this approach (6) resulted in a reasonable relation between laboratory flexural fatigue tests and cracking in the test ring. Figure 25 shows actual points of failure (individual points) superimposed over laboratory data (solid lines) for similar surface materials, for at that time data were not available for the test track pavements. The technique used for this comparison was very similar to that described for the Folsom project. The materials were characterized in the laboratory, and computations were made to estimate or predict the actual measured field behavior. By iterative procedures, a reasonable match was obtained, and the resulting pavement was used for comparison. During the field tests, surface strain was measured under repeated loads, and an average weighted strain was estimated during the period of testing. That analysis was made for the test ring completed in the spring of 1968, and, although the test sections (Table 1) were similar, actual data and weather were somewhat different from those for the 1968-69 test.

A much more detailed analysis of the Washington test track has been provided by Kingham and Kallas (7), which in many respects parallels the approach described by Monismith for the Folsom project. After the field testing was complete, pavement samples were obtained from the untraveled portion of the track. Subgrade testing had

Figure 17. Crack patterns on bottom of slabs obtained from Morro Bay project.

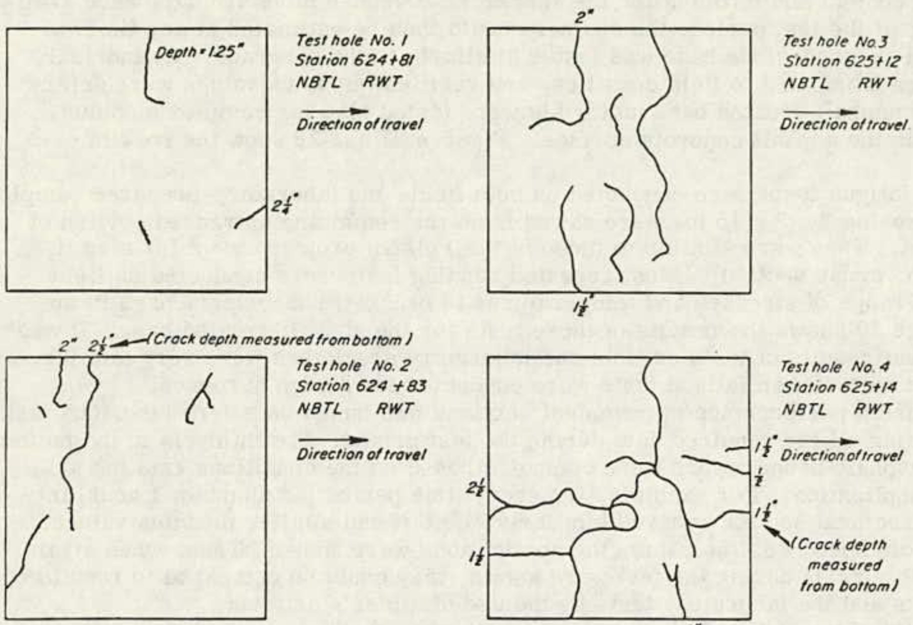


Figure 18. Loading applied by dual truck tires to Washington test track.

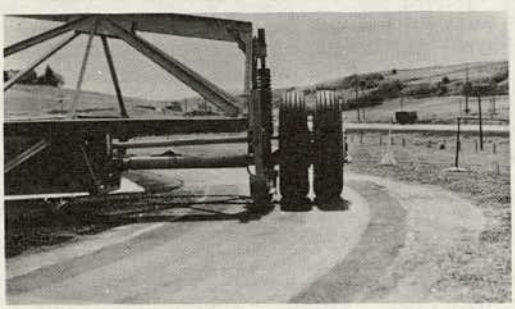


Figure 19. Typical short transverse cracks that are initial stage of more complete failure.

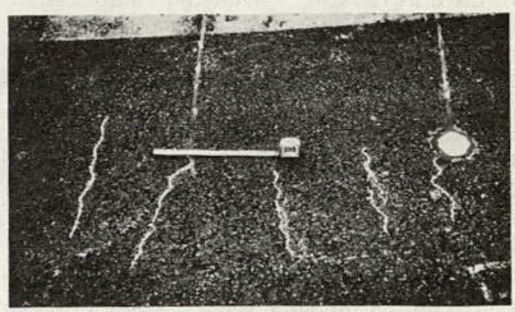


Figure 20. Further crack development and permanent deformation.

Figure 21. Ultimate form of alligator cracking covering entire trafficked area.

been completed on those materials previously (6). Figure 26 shows the relation between water content and modulus for the subgrade. Because moisture data were available throughout the test period, the stiffness could then be estimated at any time.

Untreated crushed-stone base was tested similarly to the subgrade. Cylindrical samples were fabricated to field densities, and resilient modulus values were determined. The asphalt-treated base materials were tested also for complex modulus values as was the asphalt concrete surface. Figures 27 and 28 show the results of those tests.

Flexural fatigue tests were conducted on both field- and laboratory-prepared samples. Beams measuring 3 x 3 x 15 in. were sawed from the remaining untracked portion of the pavement. They were similar to those of the Folsom project except for size (i.e., 1.5- x 1.5-in. cross section). Most repeated bending tests were conducted on field beams for a range of stresses and temperatures to bracket those experienced in the field. Figure 29 shows the results of those tests for the asphalt-treated base. It was also found that results of tests on field and laboratory specimens were very similar, an important factor when fatigue tests were conducted for design purposes.

The predicted performance of pavement sections was based on a very laborious task of accumulating all the required data during the test period. Strain levels in the bottom side of the asphalt-bound layers were computed based on the conditions existing at every load application. For example, for every time period (as short as 1 hour) in which the structural section behaved similarly, i.e., it had similar modulus values and the vehicle speed was the same, the applications were added. Then, when strain levels for all periods during the test were known, they could be compared to results of the field tests and the laboratory tests by the use of Miner's criteria.

The results of predicting fatigue failure by the procedure given above are given in Table 2. The last column in that table shows the actual count of wheel load applications to the point of cracks appearing at the surface. Both laboratory tests (stress-controlled and strain-controlled) as well as field criteria (dynamic modulus-temperature relations) were considered. Both types of laboratory tests tended to overpredict the life to initial cracking. Although comparisons were better for stress- rather than strain-controlled for the asphalt-base sections, they still were somewhat inaccurate. Comparisons for crushed stone were poor. Therefore, prediction of failure for thinner asphalt sections may be inadvisable, at least by the use of the approach described.

In summary, it would appear that prediction of pavement fatigue life is reasonable but not entirely accurate, at least with the data and techniques now available. Further work may be needed to develop criteria for pavement design with respect to prediction and prevention of fatigue distress.

SUMMARY

It would appear that much can be gained by observing the behavior of pavements in the field and through careful study of specific projects. Fatigue cracking may occur in several ways. In general, asphalt concrete surfaces on unbound aggregate bases tends to show initial distress through the development of short transverse cracks in the wheelpath. Although that is the appearance at the surface, cracks may be much more developed on the underside of the asphalt concrete.

Many pavements showing fatigue distress may have initially been affected by other factors that actually served as a catalyst to crack development. For example, cement-treated base may develop shrinkage cracks that in turn are reflected by the asphalt surface and form the beginning of a typical alligator crack system.

Most pavements tend to deteriorate rapidly once fatigue cracks develop, particularly if surface water is available to penetrate into underlying pavement materials. It is, however, often difficult to distinguish which comes first: the surface cracking that develops into a rutted or surface depression or a weak substructure that permits high strain levels to develop in the asphalt layer and thus accelerate fatigue distress. Further, it is apparent that considerable range in the service life of pavements exists after the surface has developed fatigue cracking. In some instances, the pavement structure continues to serve traffic for a considerable length of time after fatigue "failure," provided the underlying layers are not disturbed.

Figure 22. Total failure after subgrade became saturated.

Figure 23. Pumping action associated with initial short transverse cracks on pavement sections with asphalt concrete directly on subgrade.

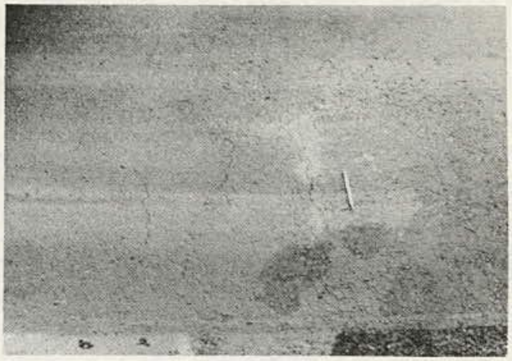


Figure 24. Well-developed crack system on underside of asphalt sections that showed little crack development at surface.

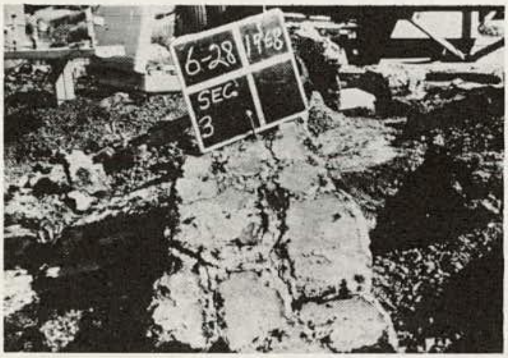


Figure 25. Laboratory fatigue data and wheel load applications.

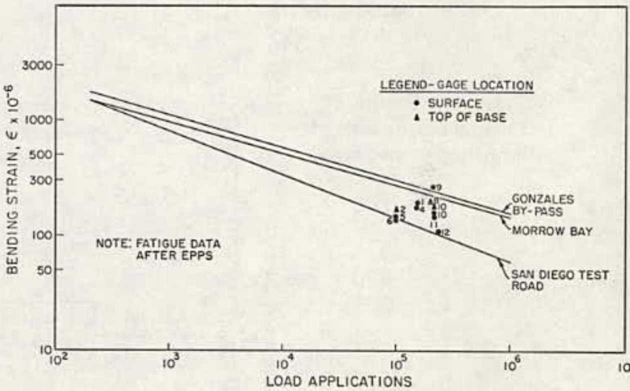


Figure 26. Variation of laboratory-measured subgrade soil stiffness.

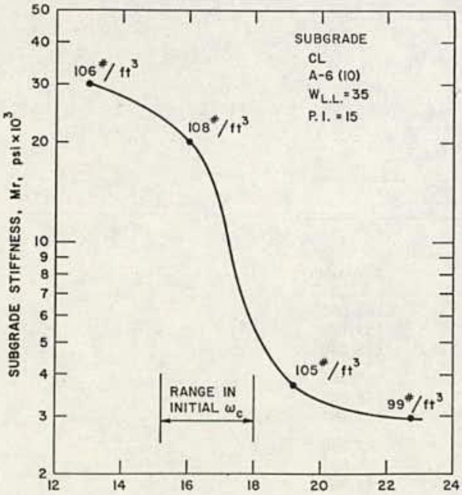


Table 1. Design of ring 4 of Washington test track.

Base	Section	Base Thickness (in.)
Sand asphalt	1	2.0
	2	4.0
	3	6.0
	4	8.0
Asphalt concrete	5	0.0
	6	2.0
	7	3.5
	8	5.0
Crushed stone	9	4.5
	10	7.0
	11	9.5
	12	12.0

Figure 27. Modulus relations for asphalt base courses.

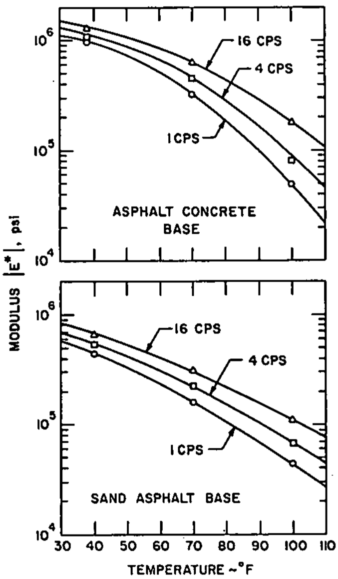


Figure 28. Modulus relations for asphalt-treated layers.

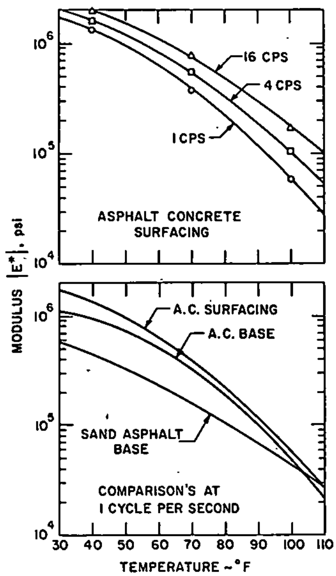


Figure 29. Results of flexural fatigue tests of asphalted-treated base.

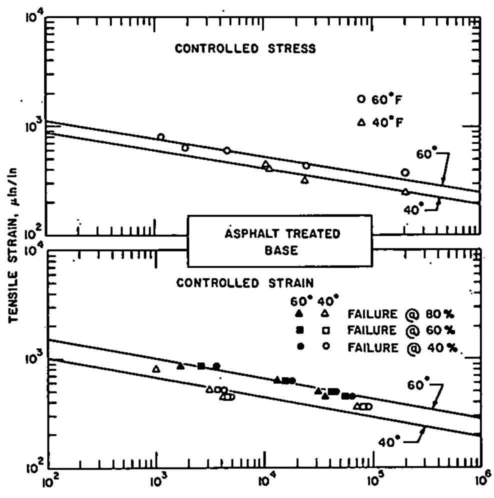


Table 2. Predicted and observed performance of pavement sections.

Base	Section	Surface and Base Thickness (in.)	Repetitions to Failure: Laboratory-Controlled Stress	Repetitions to Failure: Laboratory-Controlled Strain	Repetitions to Failure: Actual (5)
Sand asphalt	1	3 to 2	132,100	162,784*	144,660
	2	3 to 4	190,801*	190,801*	159,789
	3	3 to 6	220,189*	220,189*	175,620
Asphalt concrete	5	3.1 to 0	153,300	157,100	47,391
	6	3.1 to 2	164,800	158,137*	148,887
	7	3.3 to 3.5	174,900	170,710*	161,262
Crushed stone	9	3 to 4.5	147,000	152,700	12,000
	10	3 to 7.0	150,300	156,700	47,391
	11	3 to 9.5	153,700	158,000	48,000
	12	3 to 12	156,700	160,100	49,104

*Further analysis periods are required for the failure prediction because cycle ratios were less than 0.85 at the end of computation.

Two extensive test projects have been described that illustrate techniques for developing potential fatigue prediction procedures. One of those used an in-service pavement and the other a special test track. Both are very comprehensive in that they take into account all the variables that were feasible at the time of analysis.

The project reported by Monismith et al. was an in-service pavement that required a careful analysis of traffic loads as well as environment and materials. Characterization of the pavement structure and fatigue-life determination of the surface asphalt concrete were key factors. Once all available data were assembled, the cumulative effects of traffic on the pavement for the environmental conditions and the time period were estimated. Depending on the assumed influence of repeated traffic loads on fatigue life (in terms of strain in the asphalt-bound layer), a predicted life to failure ranged from about 9 to 12 years; the pavement was designed for 10 years according to the conventional California method. It would appear from this analysis that a reasonable estimate of design life in terms of fatigue cracking is feasible if the engineer is willing to take the time and care needed to do so.

The test pavements at the Washington State University test track were really studied from 2 standpoints: (a) observation and a limited analysis and (b) a more complete analysis based on consideration of all cumulative effects. Terrel and Krukar made a brief study in which they compared the number of load applications to initial surface cracking with failure expected by theoretically characterizing the pavements (based on modulus tests) and using fatigue data (from materials similar to that in the track). A reasonable prediction resulted but was inconclusive because of the lack of actual laboratory fatigue performance of the pavement materials used in the test track.

A more complete analysis of the Washington pavements by Kingham and Kallas probably lies somewhere between the analysis of the Folsom project and the Washington analysis discussed above. Careful characterization of the pavements and materials and also fatigue tests were provided. The traffic in terms of applied wheel loads was carefully analyzed in conjunction with changes in subgrade moisture and temperature (i.e., asphalt stiffness). Each increment of loading was accumulated as damage or distress. Predicted failure of the surface in fatigue was then compared with observed cracking in the pavements. Predictions for pavement sections with crushed-stone bases were poor; those for treated bases were somewhat better, although not so good as might have been expected.

In summary, it would appear that prediction of fatigue failure in actual pavements is feasible. However, there appears to be a basic lack of knowledge in the actual behavior of pavements under varying loads and environments. That difficulty in characterizing the pavement section, coupled with laboratory fatigue tests of the material, still leaves a gap in the understanding of all phenomena involved. Even so, more basic studies such as those described here will certainly assist the engineer to gain a better grasp of how to incorporate fatigue into a pavement design procedure.

REFERENCES

1. Phang, W. A. Four Years' Experience at the Brampton Test Road. Ontario Dept. of Highways, Rept. RR153, Oct. 1969.
2. Hicks, R. G., and Finn, F. N. Analysis of Results From the Dynamic Measurements Program on the San Diego Test Road. Proc., AAPT, Vol. 39, Feb. 1970.
3. Finn, F. N. Observation of Distress in Full-Scale Pavements. HRB Spec. Rept. 126, 1971, pp. 86-90.
4. Monismith, C. L., Epps, J. A., Kasianchuk, D. A., and McLean, D. B. Asphalt Mixture Behavior in Repeated Flexure. Univ. of California, Berkeley, Rept. TE-70-5, Dec. 1970.
5. Krukar, M., and Cook, J. C. Pavement Research at the Washington State University Test Track. Washington Dept. of Highways, Vol. 2, July 1968.
6. Terrel, R. L., and Krukar, M. Evaluation of Test Track Pavements. Proc., AAPT, Vol. 39, Feb. 1970.
7. Kingham, R. I., and Kallas, B. F. Laboratory Fatigue and Its Relationship to Pavement Performances. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972.

This paper summarizes the development of a workable design approach to analyze fatigue distress of full-depth asphalt airfield pavements. The design method uses multilayered elastic theory coupled with a limiting (critical) strain criterion developed from analysis of field results from the AASHO Road Test. The solution methodology requires that 2 specific functions be evaluated for design: the allowable traffic value function and the predicted traffic value function. The allowable traffic value represents the number of equivalent strain repetitions of a standard aircraft that a given thickness of full-depth pavement can withstand for a particular temperature and subgrade modulus combination. The predicted traffic value represents the number of equivalent aircraft strain repetitions estimated to occur during the selected design period for a given full-depth thickness. The thickness solution is obtained by the simultaneous graphical solution of both traffic value functions. Thickness design curves and aircraft equivalency diagrams are provided from which both traffic value functions are easily obtained. Design input variables needed for the thickness solution are the design subgrade modulus, mean annual air temperature, design period, and aircraft traffic forecast.

Fatigue Subsystem Solution for Asphalt Concrete Airfield Pavements

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The Asphalt Institute has recently published a revised edition of its manual, Full-Depth Asphalt Pavements for Air Carrier Airports (11). A new thickness design procedure based on multilayered elastic concepts is introduced in the manual as a workable approach to the design of full-depth asphalt airfield pavements (an asphalt pavement in which asphalt mixtures are used for all courses above the subgrade or improved subgrade). The design procedure follows the general concepts of a systems-oriented solution to the overall pavement management process in that 2 unique pavement distress modes are recognized and considered independently in the thickness solution: fatigue of the asphalt concrete layer and permanent deformation within the subgrade layer.

The detailed documentation and analysis leading to the development of the overall design method used in Manual MS-11 can be found in other reports (17, 18). The purpose of this paper is to demonstrate the applicability of the fatigue subsystem distress mode, considered in the manual, as a workable design approach. Although the paper is confined to airfield pavement design technology, the concepts presented are generally applicable as a design approach to highway pavements.

GENERAL SOLUTION

Figure 1 shows the thickness design procedure; a separate design is required for each of the 2 distress mechanisms previously described. Within each distress mode, 2 separate analyses must be accomplished. The first is termed the "allowable traffic value analysis." The solution yields the allowable number of strain repetitions of a standard aircraft (DC-8-63F) that a given thickness of full-depth pavement can sustain for a given distress type and a particular set of subgrade and environmental conditions. The second is termed the "predicted traffic analysis." The solution yields the predicted number of equivalent strain repetitions of the standard aircraft (DC-8-63F) due to the anticipated aircraft traffic mixture forecast for the service life of the airfield pavement.

The input necessary for the solution of the allowable traffic value analysis is the design subgrade modulus E_s and the mean annual air temperature t for the design location. Mean air temperature data for most design locations are readily available through various weather bureau summaries and do not present any great problem in obtaining that design factor. The design subgrade modulus of elasticity E_s is determined by 1 of 3 methods: direct measurement as described in the manual, an approximate relation to CBR, or an approximate relation to plate-bearing values.

The direct determination is made by a repeated load triaxial compression test detailed in the manual. The commonly used method of determining subgrade strength for airport pavements, the California bearing ratio (CBR), makes use of the relation E_s = 1,500 CBR. AASHO Test Method T 193 or ASTM Test Method D 1883 is used for the CBR test. Samples are compacted according to method B or D of ASTM Test Method D 1557 (AASHO Test Method T 180).

The requirements for the plate-bearing test are detailed in Manual MS-10. The relation to E_s is given in the design manual. The test is performed on compacted subgrade in trial sections prepared exactly as it will be done for the finished pavement.

Knowledge of those 2 variables allows one to select the appropriate design chart in Manual MS-11 for the distress mode considered. A schematic illustration of a typical allowable traffic curve is shown in Figure 2a for the fatigue mode of distress for a particular E_s and t. The curve represents the allowable number of strain repetitions N_a of the standard aircraft that a given full-depth pavement thickness can sustain in fatigue.

For the predicted traffic analysis, the requisites for solution are a prediction of the specific aircraft types and associated traffic levels of each aircraft within the design period evaluated. From that information, the predicted traffic curve can be developed from aircraft equivalency diagrams found in Manual MS-11. That curve is shown in Figure 2b and represents, at a given asphalt concrete thickness T_A, the number of equivalent strain repetitions of the standard aircraft caused by the anticipated aircraft mixture within the design life considered. The thickness solution for the fatigue subsystem is determined by the intersection of the 2 curves shown in Figure 2c. That is, at the design T_A value, the predicted number of equivalent strain repetitions due to the mix is equal to the allowable number of fatigue strain repetitions of the standard aircraft that the pavement thickness can sustain for the given subgrade and environmental conditions.

SUMMARY OF FACTORS CONSIDERED

The mechanics of the general solution provided in Manual MS-11 for either distress mode are quite straightforward. Required input variables are confined to those normally needed for most pavement design problems. Hence, the basic approach to the fatigue solution presented provides a workable system that can be used with identical input parameters required for most empirical design approaches.

Factors considered in the allowable traffic analysis, which has inputs of E_s and t, are as follows:

E_n, ν_n, and h_n = stress and strain states evaluated through multilayered theory and used to determine critical horizontal tensile strain values at the bottom of asphalt-bound layer;

N_f-ϵ = failure criterion developed from actual pavement performance studies;

$\sum(n_i/N_f)$ = 1, Miner's hypothesis, used as cumulative fatigue damage model;

E_1-t-f = asphalt concrete characterization, average dynamic modulus-temperature-frequency response of dense-graded asphalt concrete mixtures obtained from laboratory tests;

f = design frequency or rate of loading (f = 2 cps) selected as representative loading rate of dual tandem standard aircraft gear at taxiway speed of 10 to 20 mph;

m_q = monthly modulus distribution, which accounts for the predicted monthly distribution of asphalt concrete modulus for design environmental conditions (prediction based on field pavement temperature studies); and

E_s = subgrade modulus, which accounts for variation in subgrade support due to effects of anticipated freeze-thaw conditions of subgrade.

Factors considered in the predicted traffic analysis, which has inputs of j, p_j, and L_d, are as follows:

j = aircraft type, which accounts for specific aircraft types in the anticipated aircraft mix;

p_j = number of passes, which considers predicted traffic volume levels for each aircraft type in the mix;

L_d = design life, which allows for design solution to be determined for any desired service life;

S, P, and p_t = aircraft characteristics, which includes the unique load distributing characteristics of each aircraft type in the mix, e.g., gear type, tire spacings, load per tire, and tire pressure;

X_j = point of load application, which accounts for differences between aircraft types in the lateral distance between aircraft and main gear centerline; and

σ_w = lateral wander effect of channelized taxiway traffic conditions.

The remaining sections of this paper discuss how those fundamental characteristics have been incorporated into the solution of the fatigue subsystem for full-depth airfield pavements.

ALLOWABLE TRAFFIC ANALYSIS

Allowable Traffic (Fatigue) Expression

The development of the design fatigue equation (allowable traffic equation) was accomplished through a series of multiple regression equations.

1. The monthly pavement temperature frequency distribution equation characterizes the mean and standard deviation of monthly pavement temperature distribution from mean monthly air temperature data. The equation is based on correlation studies from field tests in Arizona, Maryland, and New York and is valid only for thick (>10.0 in.) asphalt concrete layers.

$$\overline{t}_{p_m} = B_0 \left(\overline{t}_{a_m}\right) + B_1$$

$$\sigma = B_2 \tag{1}$$

where

$\overline{t}_{p_m}$ = mean monthly pavement temperature;

$\overline{t}_{a_m}$ = mean monthly air temperature;

σ = standard deviation of pavement temperature; and

B_0, B_1, and B_2 = constants with values 1.05, 5, and 5 respectively.

2. The asphalt concrete modulus-temperature relation equation predicts the dynamic modulus of a dense-graded asphalt concrete mix for a given temperature. The rate of loading applicable for the equation is f = 2 cps and is based on regression analysis of laboratory dynamic modulus test results.

$$E_1 = \frac{K_0}{K_1^{q^{d_1}}} \tag{2}$$

where

q = pavement temperature; and
K_0, K_1, and d_1 = constants with values 3.8×10^5, 1.0046, and 1.45 respectively.

3. The multilayered principal asphalt concrete tensile strain for standard aircraft equation predicts the maximum principal tensile strain at the bottom of the full-depth pavement due to the standard aircraft (DC-8-63F) at the conditions of h_1, E_1, and E_2. Poisson's ratio values used to develop the equation are 0.40 and 0.45 for asphalt concrete layer and subgrade soil respectively.

$$\epsilon = \frac{M_0}{h_1^{A_1} E_1^{A_2} E_2^{A_3}} \tag{3}$$

where

ϵ = maximum principal tensile strain at bottom of asphalt concrete layer due to DC-8-63F aircraft;
h_1 = thickness of full-depth pavement;
E_1 = modulus of asphalt concrete layer;
E_2 = subgrade modulus of elasticity; and
M_0, A_1, A_2, and A_3 = constants with values 1.086×10^3, 1.19967, 0.66866, and 0.320867 respectively.

4. The fatigue criteria equation predicts the relation between the allowable number of tensile strain repetitions to failure and the associated tensile value for a given pavement temperature. Criteria were developed from analysis of AASHO Road Test data for thick asphalt concrete layers.

$$N_{f_q} = ab^{q^{d_1}} \left(\frac{1}{\epsilon}\right)^c \tag{4}$$

where

N_{f_q} = number of allowable tensile strain repetitions at pavement temperature q;
ϵ = as defined in Eq. 3; and
a, b, c, and d_1 = constants with values 1.86351×10^{-17}, 1.01996, 4.995, and 1.45 respectively.

The major relation is that noted by Eq. 4, which expresses the fatigue criterion for a typical asphalt concrete mixture. Figure 3 shows that criterion in terms of mix temperature defined in accordance with Eq. 2. The equations given above coupled with Miner's hypothesis for cumulative damage on a monthly basis allowed the development of the following basic fatigue equation:

$$h_1 = T_A = \left[\frac{N_f M_0^c}{12aK_0^{A_2 c}} \sum_{i=1}^{12} \sum_{j}^{J} \left(\frac{m_{q_{ij}} K_1^{A_2 c q_j^{d_1}}}{b^{q_j^{d_0}} E_2 i^{A_3 c}} \right) \right]^{\frac{1}{A_1 c}} \tag{5}$$

Equation 5 yields the required thickness of full-depth asphalt concrete T_A for the fatigue distress mode for a given allowable number of strain repetitions N_f, subgrade modulus for the i th month $E_2 i$, and monthly temperature data. $m_{q_{ij}}$ represents the monthly frequency of the i th month that the pavement temperature q will be within a certain temperature interval, $q_{ij} \pm \alpha$. The subscript j denotes the j th pavement temperature interval, 10 F. Therefore, even though mean air temperatures are used as input for any given month, the solution is based on a predicted normal distribution (frequency) of pavement temperatures for that month. A more thorough development and explanation of the equation is given in an earlier report (17).

Figure 1. Airport pavement thickness design procedure.

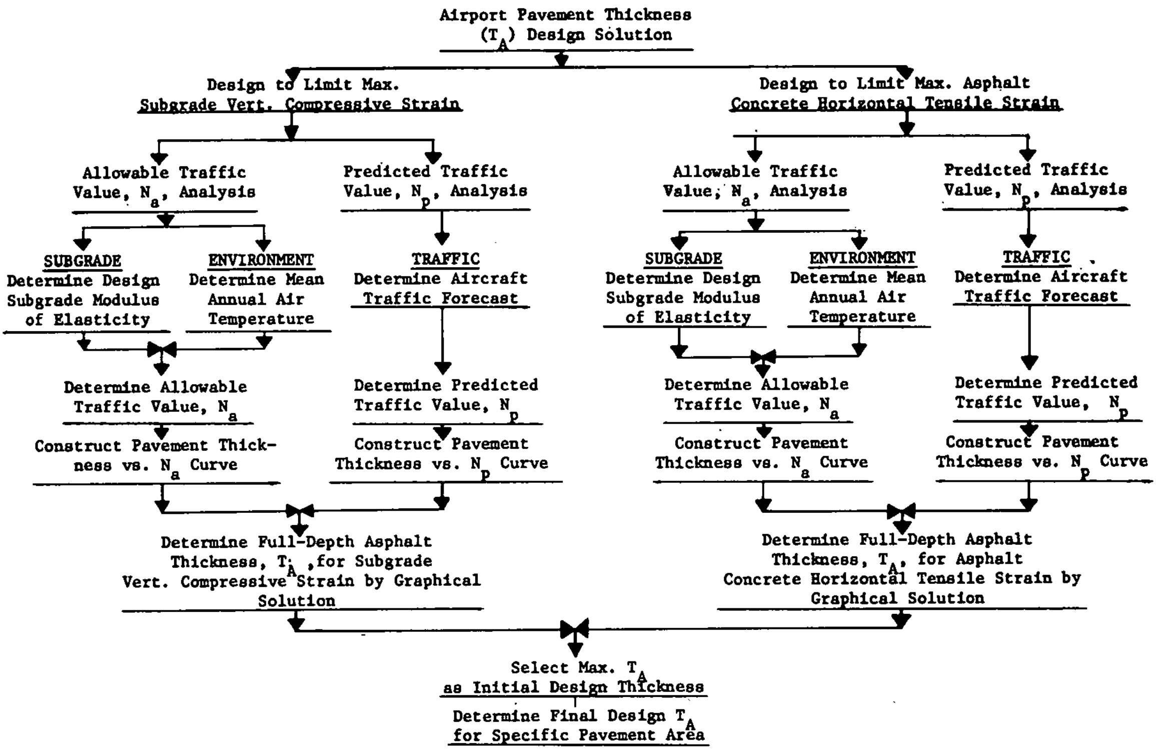


Figure 2. Schematic solution for T_A design.

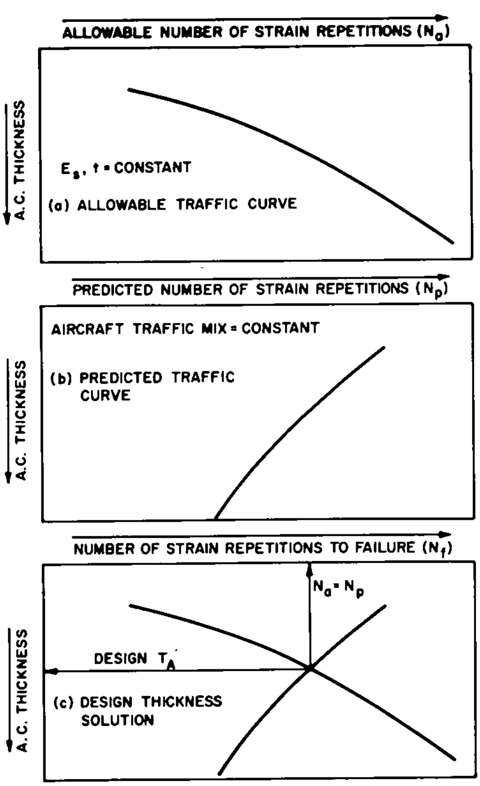
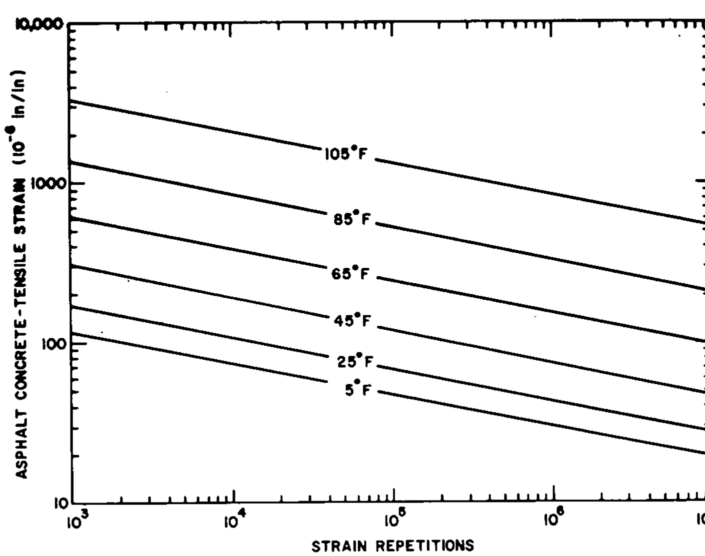


Figure 3. Allowable asphalt concrete tensile strain criteria.



Environmental Effects

The major significance of the model developed is that it allows the effects of monthly variations of both predicted temperature distribution and subgrade support values to be investigated. Hence, through use of this expression, a comprehensive study was conducted to ascertain certain environmental effects on design thickness requirements for full-depth pavements.

A full-depth asphalt pavement possesses load-distributing characteristics similar to classical flexible and rigid pavement systems. The behavior is primarily influenced by the modulus of the asphalt layer, which in turn is significantly dependent on the pavement temperature. The extreme dependency of the elastic layered response (stress and strain) to the modulus logically implies that identical full-depth pavements should perform differently in differing environmental situations.

As an example of this aspect, Figure 4 shows a comparison of predicted yearly frequency distribution of moduli for thick asphalt concrete pavements placed in Brownsville, Texas, and in Winnipeg, Canada. The difference in the predicted distributions is apparent. For example, the mean moduli values are about 250,000 psi for Brownsville and 1,100,000 psi for Winnipeg. The percentage of the year the moduli will be above 500,000 psi is about 5 percent in the warm Texas climate and about 75 percent in Canada. In concept, if placed in the colder climate, the full-depth asphalt concrete modulus will approach values quite typical of that of a rigid pavement condition during a larger relative proportion of the year. Conversely, the yearly percentage when the asphalt concrete modulus will be in a more flexible state if placed in the Texas location should also be readily observable. It is subsequently felt that, for thick asphalt concrete pavements, the effect of environment on structural design considerations is as important a design variable as subgrade support and traffic.

Predicted Monthly Damage Percentage

The design equation developed for the fatigue subsystem allowed the determination of the predicted monthly damage distribution for any environmental input; monthly temperature and subgrade modulus values are used in the predictions. Figure 5 shows the predicted fatigue in Swea City, Iowa. Monthly air temperature values for the site were translated into monthly pavement temperature frequencies from Eq. 1. Monthly subgrade modulus values were selected based on the anticipated duration of freezing and thawing developed from a freezing index analysis of the site. For this case, a normal modulus E_{2_n} of 6,000 psi was selected with a minimum modulus of 0.4 E_{2_n} during the thaw period and a maximum subgrade modulus of 4.0 E_{2_n} during the freeze. Monthly moduli for the other months were selected by assuming that a linear relation existed between the normal, frozen, and thawed states.

Also shown in Figure 5 is the distribution of actual pavement failures (main factorial study) observed by month at the AASHO Road Test. The temperature and subgrade modulus distribution for the predicted damage location, although not identical, is quite similar to that found at Ottawa, Illinois, the site of the AASHO Road Test. Even though the actual distribution of pavement failures is for flexible (granular base) pavements and the predicted values are based upon full-depth concepts, the general agreement in magnitude and trends is felt to be quite encouraging and supports the basic model used in the analysis.

Effect on Full-Depth Thickness

The effect of both variable monthly pavement temperature and subgrade values on full-depth thickness was determined by an analysis of more than 130 locations in North America through a computerized solution of the fatigue equation developed. Figure 6 shows the results of that study. Data are plotted by the mean annual air temperature of each site investigated against a thickness adjustment factor T_F. That factor is the ratio of the thickness obtained by a monthly cumulative damage model for the variable monthly conditions to the thickness obtained at a constant subgrade modulus and an arbitrarily selected constant pavement temperature of 40 F.

Figure 4. Yearly frequency distributions of asphalt concrete moduli for 2 locations.

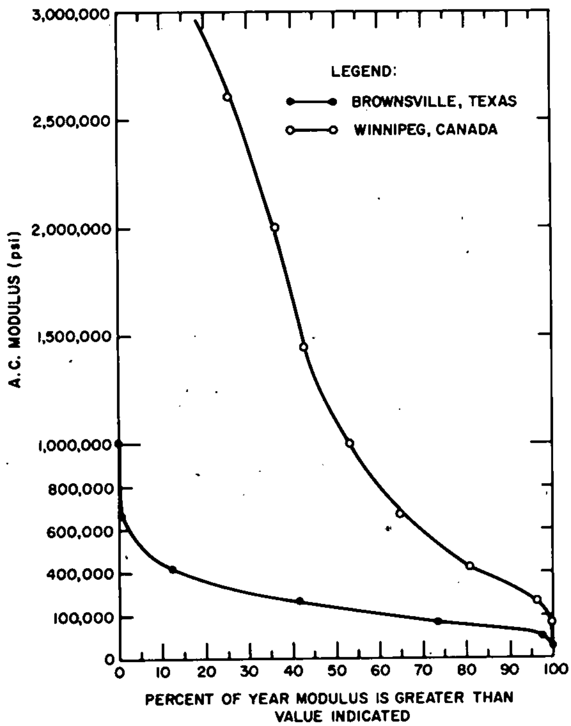


Figure 5. Predicted monthly fatigue damage to actual monthly failure distribution of main experiment.

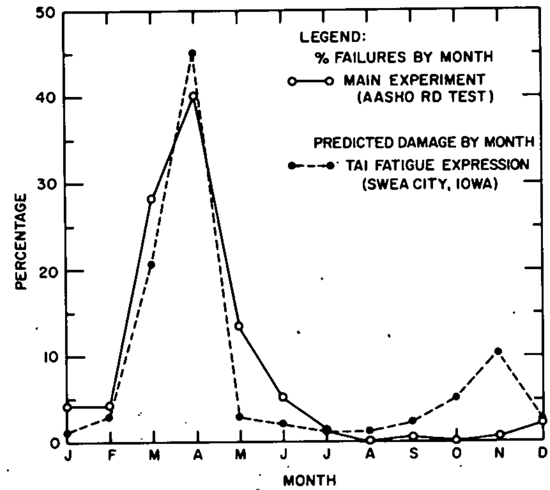
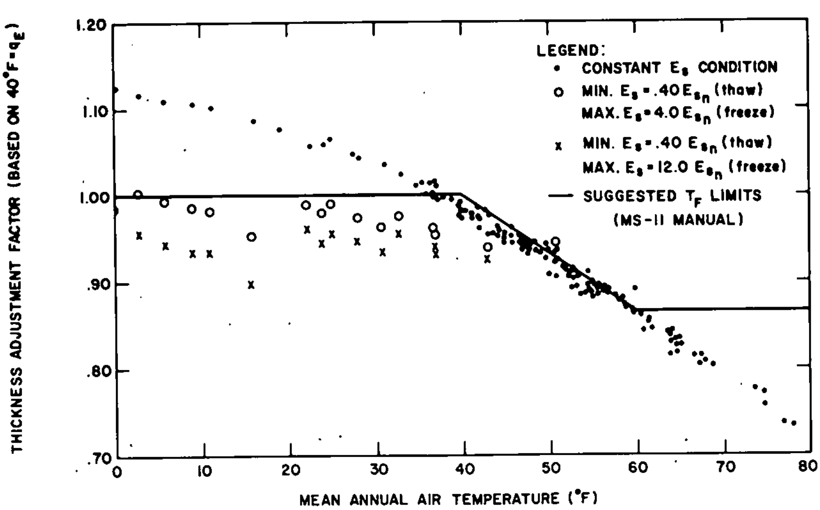


Figure 6. Suggested thickness adjustment factors for asphalt concrete tensile strain analysis.



Analyses were based on the assumption that the subgrade modulus would be constant throughout the year and would also vary because of freeze-thaw conditions in accordance with the procedure previously described. The results of that study show that, for the constant yearly subgrade modulus condition, greater full-depth thicknesses are required for colder conditions because of fatigue cracking within the asphalt-bound layer. However, when variable subgrade conditions are introduced to take into account monthly variations in modulus due to anticipated freeze-thaw states, thickness requirements generally occur below that shown for a constant subgrade condition corresponding to a mean annual air temperature of approximately 40 F.

The explanation for that occurrence is that during cooler periods, although tensile strains are more critical, the probability of a very stiff subgrade due to frozen conditions may be large. Therefore, the large frozen subgrade modulus results in extremely smaller tensile strains than would be obtained at the normal or constant subgrade modulus. Dempsey and Marek also found that to be conceptually true in their theoretical study of the bidaily radial stress and strain predictions of several test sections of the AASHO Road Test (10). Based on a combined heat-transfer model for evaluating the temperature regime and frost-line position in the pavement system and an elastic layer model for stress and strain determination, they found that during most of the frozen subgrade period the horizontal state of stress and strain existing at the asphalt concrete base interface was either close to zero or actually transferred into a compression phase.

Based on that analysis, the T_F limits selected for use in Manual MS-11 are also shown in Figure 6. The lower limit of the adjustment factor used (T_F = 0.866) corresponds to a mean annual air temperature of approximately 60 F. That arbitrary limit was selected primarily on the basis of a conservative safety factor.

Effective Temperature Concept

One of the important features of the developmental work leading to the fatigue subsystem design in Manual MS-11, and one with extreme practical applications, is that of the effective temperature concept (17). An effective pavement design temperature q_e is defined as the unique pavement temperature at which the allowable number of strain repetitions to failure, obtained by using the variable monthly design equation, is equal to the failure repetitions obtained from Eq. 4 at $q = q_e$; the tensile strain value for the design load is calculated from multilayered theory at an asphalt concrete modulus corresponding to a temperature of q_e.

Figure 7 shows the relation between the mean annual air temperature for the design location and the effective pavement temperature to be used for the fatigue criterion. Figure 8 shows the relation between the mean annual air temperature and the effective asphalt concrete modulus to be used in the multilayered analysis of the tensile strain.

Example Problem

The significance of using the thickness adjustment factor T_F and the effective temperature q_e methods to obtain an identical solution to the fatigue problem is demonstrated in the following example.

A full-depth pavement is to be analyzed for fatigue of the asphalt pavement layer. The subgrade modulus is 7,500 psi. The design aircraft is a DC-8-63F (358,000-lb max gross weight). Poisson's ratios of the asphalt and subgrade layers are 0.40 and 0.45 respectively. The mean annual air temperature at the design location is 60 F.

In the thickness adjustment factor analysis method, the basic fatigue criterion is always defined as q = 40 F by fatigue Eq. 4. Likewise, the asphalt concrete modulus used to evaluate the tensile strain is always 1,450,000 psi. A multilayered analysis using the Shell BISTRO computer solution for the design aircraft and h_1 = 20 in., E_1 = 1,450,000 psi, ν_1 = 0.40, E_2 = 7,500 psi, and ν_2 = 0.45 resulted in a maximum principal tensile strain of 137 μin./in. Based on q = 40 F and $\epsilon = 137 \times 10^{-6}$ in./in. from Eq. 4, the number of strain repetitions to failure for the fatigue distress mode is approximately 20,000. The design mean annual air temperature is 60 F, and the T_F value for that location is T_F = 0.866. Therefore, the acutal full-depth thickness that would sustain 20,000 repetitions to failure would be $h_a = h_1(T_F) = 20(0.866) = 17.32$ in. In summary, a full-depth pavement 17.32 in. thick placed in a design locale having a mean annual air temperature of 60 F would be able to withstand approximately 20,000 strain repetitions of a DC-8-63F aircraft.

In the effective temperature concept analysis procedure, data shown in Figures 7 and 8 are used to determine the effective pavement temperature for use in the fatigue equation and the effective asphalt concrete modulus to be used in the multilayered analysis of the tensile strain. For a design air temperature of 60 F, $q_e = 62$ F (Fig. 7). The effective asphalt concrete modulus E_e is approximately 600,000 psi. For comparable conditions of the problem solved by the T_F method, the multilayered input used is $h_1 = 17.32$ in., $E_1 = 600{,}000$ psi, $v_1 = 0.40$, $E_2 = 7{,}500$ psi, and $\nu_2 = 0.45$. The multilayered analysis of the DC-8-63F aircraft with the Shell BISTRO computer solution results in a tensile strain value of 280 μin./in. Therefore, the allowable number of strain repetitions to failure by fatigue Eq. 4 with $q_e = 62$ F and $\epsilon = 280 \times 10^{-6}$ in./in. is 19,513.5 or 19,500. That value compares quite well with the 20,000 strain repetitions predicted by the T_F method.

Typical Fatigue Curves

Based on the concepts presented, full-depth design curves for the fatigue distress mode developed by the T_F analysis are presented in Manual MS-11. A typical set of those curves is shown in Figure 9. The curves already incorporate the required thickness adjustment factor. The number of strain repetitions refers to the standard aircraft.

As noted previously, the only input needed for the solution of the allowable traffic curve is the mean annual air temperature and design subgrade modulus. A typical allowable traffic curve (fatigue mode only) is shown in Figure 10 for $E_s = 15{,}000$ and $t = 50$ F. The T_A values were obtained for the various strain repetition curves at an $E_s = 15{,}000$ psi from the 50 F design curve shown in Figure 9.

PREDICTED TRAFFIC ANALYSIS

Predicted Traffic (Fatigue) Expression

The allowable traffic curves previously presented express the thickness of full-depth pavement necessary to prevent the fatigue mode of distress for any desired number of strain repetitions of a standard aircraft. The aircraft selected as the standard for use in Manual MS-11 is the DC-8-63F (358-kip maximum gross weight). The DC-8-63F, therefore, is the airfield design counterpart of the familiar 18-kip axle load used as the standard design vehicle in many highway pavement design methods. Because a standard aircraft is used, the effects of differing aircraft types on the cumulative fatigue damage are incorporated into the fatigue subsystem through the use of aircraft equivalency factors, which indicate the relative damage caused by any particular aircraft to that caused by the standard aircraft.

Unlike most highway considerations, the analysis of mixed airfield traffic possesses 3 important characteristics that distinguish it from the typical equivalency analysis commonly used in highway applications.

1. Each aircraft type has its own unique load-transmitting system; that is, the family of aircraft main gears comprises differing tire spacings, gear types, locations, gear loads, and tire pressures. That contrasts with the typical grouping of most highway vehicles in that the primary variable is a rather restricted range of axle loads, while tire pressures, spacings, and gear types vary over a relatively small range and may, for the most part, be considered as constants among vehicular types.
2. In highway pavements, the lateral wander effect of moving vehicles is extremely small and can be effectively assumed to be nonexistent. This implies that one pass, or movement, of a highway vehicle will cause one strain (stress or damage) repetition at the critical design location. However, airfield design studies have indicated that lateral distribution of aircraft should be considered for a more accurate estimate of the traffic analysis (4, 13, 14). Even for channelized taxiway conditions, N passes of an aircraft may not yield N strain repetitions at the maximum damage location but rather some defined percentage of that value.

Figure 7. Mean annual air temperature and effective pavement design temperature for establishing fatigue criterion.

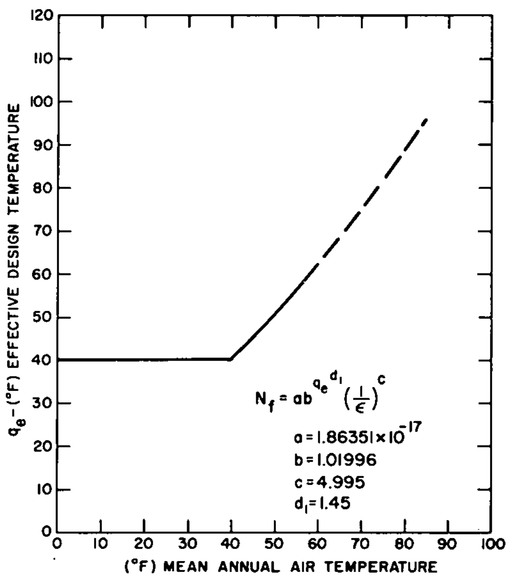


Figure 8. Mean annual air temperature and effective asphalt concrete modulus to be used in multilayer analysis for evaluating critical tensile strain.

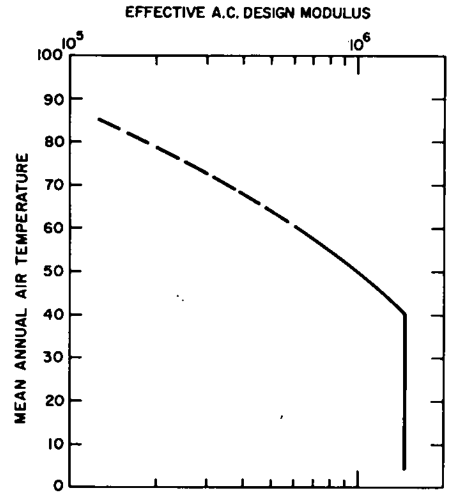
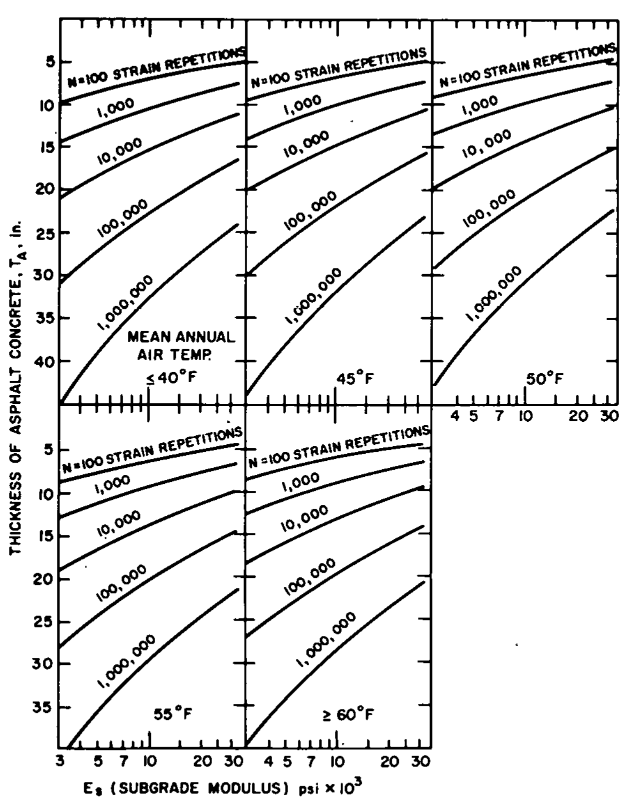


Figure 9. Design curves for asphalt concrete fatigue distress.



3. The final difference between the 2 types of analysis is the mean lateral location of the point of load application. In highways, the mean location is slightly more than 2 ft from the pavement edge and is assumed to be equal for all vehicular types. In airfield analysis, the distribution of aircraft is centered about the pavement centerline (taxiway or runway). For that situation, it is obvious that the point of maximum damage due to the main gear loads will differ significantly among the aircraft types because of inherent differences (6 to 19 ft) in the transverse spacings between aircraft and main gear centerlines. Consequently, 2 aircraft having those pronounced differences in gear locations and moving directly along the taxiway centerline would not cause strain or damage repetitions at the same location.

The aircraft traffic mixture analysis used in Manual MS-11 incorporates each of the characteristics. That results in a method that allows the determination of both the maximum anticipated number of equivalent DC-8-63F strain repetitions and the transverse taxiway location where that maximum damage will occur.

The design expression developed to account for the predicted mixture of aircraft traffic in the fatigue design of the critical taxiway is

$$n_e = \max \sum_{j=1}^{J} p_j \, f_{jx} \, F_{jh} \tag{6}$$

where

- n_e = the maximum (design) equivalent DC-8-63F strain repetitions predicted across (transverse) the taxiway;
- p_j = number of taxiway passes of the j th aircraft;
- f_{jx} = transverse distribution factor that relates the frequency percentage of tensile strain repetitions at the x th lateral taxiway interval (1 ft wide) due to the j th aircraft;
- F_{jh} = equivalent aircraft damage factor of the j th aircraft on a full-depth pavement h thick (in this paper, the factor relates the equivalent damage only due to the fatigue mode of distress); and
- h = constant.

A detailed development of Eq. 6 can be found in another report (18).

Figure 11 shows, in schematic form, the basic concepts behind the equivalent damage equation: the frequency of strain repetitions along the taxiway for the 2 aircraft and the number of strain repetitions occurring along the taxiway due to p passes of each aircraft type. The product of the strain repetitions at a given interval with the equivalent damage factor yields the number of equivalent repetitions of the standard aircraft along the taxiway for each aircraft. Summing the equivalent repetitions, at a given interval for both aircraft, gives the cumulative distribution of equivalent repetitions across the taxiway. That is shown by the diagram at the bottom of the figure. The maximum value and its corresponding location yield the desired n_e value of the design equation.

In the design expression, the aircraft passes are data input determined from an analysis of the anticipated aircraft traffic for the airfield in question. The solution, therefore, depends on the establishment of the transverse distribution factor f_{jx} and the equivalent aircraft damage factor F_{jh} for the specific aircraft in the mix.

Characteristics of Design Expression

Transverse Distribution Factor f_{jx}

The f_{jx} factor represents the frequency of strain repetitions at the taxiway interval; x = (a - b) for the specific aircraft in question. That factor accounts for the lateral wander distribution σ_w of channelized taxiway traffic and the lateral distance between aircraft and main gear centerlines X_j for each specific aircraft type. The fundamental concept behind the f_{jx} factor is that the lateral distribution of the maximum principal tensile strain is normally distributed and can be characterized by $u = \pm X_j$, σ_w for all aircraft.

Studies have shown that, for channelized taxiway traffic, σ_w is approximately 40 in. or about 3.5 ft. Because X_j is unique for each aircraft type, the continuous normal distribution defined by those parameters can be translated into discrete frequency values f′, taken at 1-ft-wide taxiway intervals by the probability equation

$$f'_{(a-b)} = P_r\ (a < x < b) = P_r\ (x > a) - P_r\ (x > b) \tag{7}$$

Those discrete frequency values would be equivalent to the f_{jx} value provided the width of the distressed region (strain width) is exactly 1 ft. That, however, is not the case for almost all aircraft. As a simplifying assumption, the effective width of the principal horizontal tensile strain at the bottom of the asphalt layer was selected to be equal to 2S (where S is the center-to-center spacing between outermost tires, in transverse direction, of the aircraft main gear). That assumption gives very excellent agreement with a more detailed study conducted by Deacon (4) using the actual variable transverse strain distribution for multiwheel aircraft. A further discussion of that comparison is given in another report (18).

Use of the foregoing assumption, coupled with the normal distribution characterization parameters previously defined, allowed the determination of f_{jx} factors for 22 selected jet aircraft. Table 1 gives those values at selected taxiway intervals for the aircraft noted. The practical interpretation of the table is that, if 100 passes of a DC-9-41 aircraft occurred on a taxiway, 21 tensile strain repetitions (DC-9-41) would occur at a location ±12.5 ft from the taxiway centerline.

Equivalent Damage Factor F_{jh}

The F_{jh} value indicates the unit damage caused by the j th aircraft relative to the unit damage caused by the standard s aircraft. For the fatigue distress mode, the F_{jh} factor is R^c, where $R = (\epsilon_{tj}/\epsilon_{ts})$, the ratio of the maximum principal tensile strain of the j th aircraft relative to the maximum principal tensile strain of the standard aircraft evaluated under identical multilayered pavement conditions, and c represents the slope of the allowable fatigue criterion noted in Eq. 4.

The maximum principal tensile strain in the asphalt-bound layer in a full-depth pavement is a function of

$$\epsilon_t = f(j,\ h_1,\ E_1,\ E_2,\ v_1,\ v_2) \tag{8}$$

For a given aircraft type and constant Poisson's ratio values of 0.40 and 0.45 for the asphalt and subgrade layer respectively, the tensile strain and, hence, R value are analytically functions of the thickness h_1 and layered moduli values E_1 and E_2.

A multilayered study was undertaken to ascertain to what degree, if any, those variables influenced the R_j value. Figure 12 shows a partial summary of the R_j results developed. The major conclusion that can be drawn from that study is that, for all practical purposes, the F_j factor is a function only of the asphalt concrete pavement thickness h_1. Because the particular set of moduli values used for both layers appears to be insignificant, the F_{jh} values for fatigue distress are also applicable for layered systems comprising more than 2 layers. Similar conclusions have also been independently obtained by Deacon (4).

F_{jh} values for the fatigue mode of distress for the 22 aircraft previously given in Table 1 were computed from multilayered theory by the use of the Shell BISTRO computer program. Individual aircraft gear characteristics were used for each layered input solution to determine the maximum principal tensile strain value. Constant layer moduli of E_1 = 500,000 psi and E_2 = 7,500 psi were selected for all problems analyzed. However, to take into account the effect of varying stress pulses (load frequency) on E_1 at a constant temperature, triple tandem aircraft gears were analyzed at an E_1 = 420,000 psi (approximately 1 cps), and dual tire aircraft gears were analyzed at an effective E_1 = 600,000 psi (approximately 4 cps). The standard aircraft has a dual tandem gear configuration corresponding to a frequency of about 2 cps for the design taxiway velocity chosen. Table 2 gives the F_{jh} values for the selected aircraft. The

Figure 10. Allowable traffic value curve for asphalt concrete horizontal tensile strain.

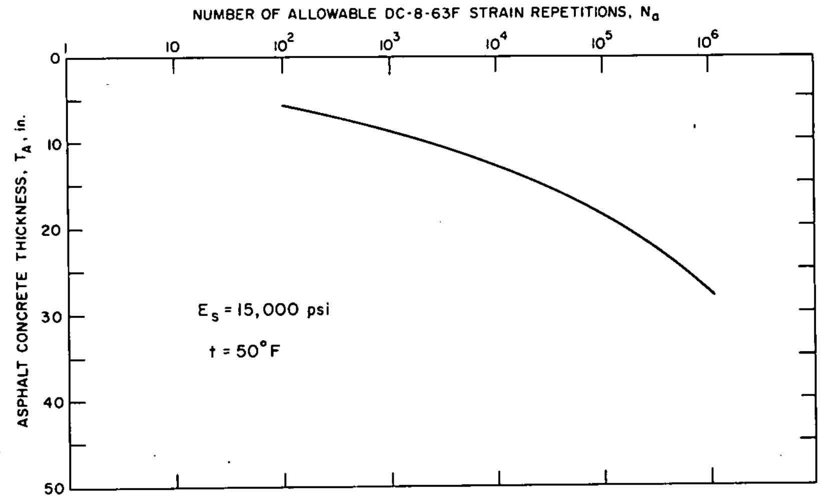
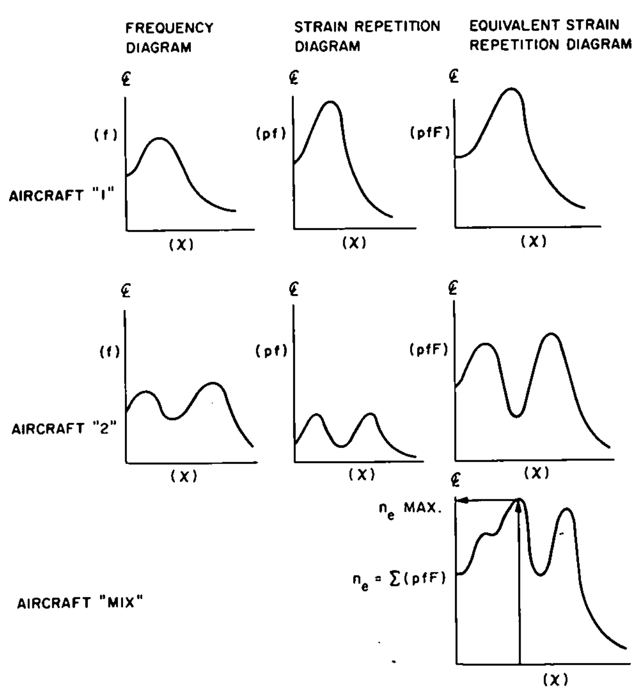


Table 1. f_{jx} values for various intervals from taxiway centerline.

Aircraft	0-1 Ft	4-5 Ft	8-9 Ft	12-13 Ft	16-17 Ft	20-21 Ft	24-25 Ft	Max
B-747	0.45	0.68	0.62	0.45	0.68	0.59	0.18	0.71
B-747F	0.45	0.68	0.62	0.45	0.68	0.59	0.18	0.71
B-707-320C	–	0.15	0.52	0.60	0.23	0.02	–	0.64
B-707-120B	–	0.15	0.52	0.60	0.23	0.02	–	0.64
B-720	–	0.11	0.44	0.51	0.18	0.01	–	0.54
B-727-200	0.05	0.28	0.62	0.47	0.12	–	–	0.66
B-737-200C	0.05	0.30	0.56	0.30	0.04	–	–	0.56
CV-990	–	0.14	0.42	0.36	0.09	–	–	0.46
CV-880M	0.01	0.17	0.44	0.32	0.06	–	–	0.46
L-500	0.01	0.22	1.00	1.66	1.24	0.36	0.02	1.66
L-1011-8	–	–	0.06	0.38	0.71	0.49	0.11	0.71
L-1011-1	–	–	0.06	0.38	0.79	0.72	0.27	0.83
DC-10-30CF	–	–	0.09	0.44	0.81	0.67	0.23	0.84
DC-10-10	–	–	0.09	0.44	0.81	0.67	0.23	0.84
DC-8-63F	–	0.15	0.48	0.48	0.15	–	–	0.56
DC-8-61	–	0.15	0.48	0.48	0.15	–	–	0.56
DC-9-41	0.07	0.29	0.46	0.21	0.02	–	–	0.46
DC-9-15	0.07	0.29	0.46	0.21	0.02	–	–	0.46
Concorde	–	0.03	0.24	0.46	0.24	0.03	–	0.46
BAC-1-11-500	0.14	0.36	0.42	0.14	–	–	–	0.46
VIS-810	–	0.03	0.21	0.33	0.15	0.01	–	0.33
SE-210-6R	0.03	0.19	0.36	0.19	0.02	–	–	0.36

Figure 11. Schematic solution of basic equivalent damage equation.



interpretation of the data given in Table 2 is that an L-1011-8 aircraft is about 4 times (3.843) as damaging in the fatigue mode of distress as the DC-8-63F on a 20-in. full-depth pavement.

Methods of Solution

Manual Version

For a given aircraft type, the predicted traffic equation is

$$n_{e_x} = p_j f_{jx} F_{jh} \tag{9}$$

That equation, in logarithmic form, is linear with a slope of 1, and the intercept is defined by a unique combination of j, h, and x.

$$\log n_{e_x} = \log p_j + \log C \tag{10}$$

where $C = f_{jx} F_{jh}$.

The form of Eq. 10, which is valid for all aircraft, enables aircraft equivalency diagrams to be developed that relate the number of equivalent DC-8-63F tensile strain repetitions to the number of taxiway passes of the aircraft in question. Each relation defined is for a unique interval x and asphalt concrete thickness h_1.

That is the method presented in Manual MS-11 for the solution of the predicted traffic equation. A typical fatigue equivalency diagram from the manual is shown in Figure 13 for the L-1011-8 aircraft. Similar diagrams for all 22 aircraft given in Table 1 are presented for asphalt concrete thicknesses of 10, 30, and 50 in. A simple solution to determining the predicted traffic values is given in Manual MS-11; work sheets are included for use in a step-by-step solution.

Computerized Version

Although the equivalency diagrams previously discussed can be used for any anticipated aircraft mixture, their use becomes quite laborious when many different aircraft types, traffic volumes, and design periods are evaluated. To alleviate that situation, a computer program was developed for the solution of any aircraft mixture forecast within the service life of the full-depth pavement. That program, itself, forms one of the subroutines of The Asphalt Institute's full-depth airfield design computer program.

The only input required by the design engineer is a traffic forecast of the aircraft types and estimated number of taxiway passes of each aircraft anticipated during given time intervals (5-year period) up to the service life. The program computes the distribution of the standard aircraft strain repetitions along specified taxiway intervals for asphalt concrete thicknesses of 10, 20, 30, and 40 in. for both modes of distress and determines the maximum design strain repetition value and transverse location of that maximum damage for the input data. An example illustrates the use, results, and interpretations of the program.

Table 3 gives a typical format entry of the input data. The numbers represent the anticipated number of taxiway passes within a 5-year period for each aircraft. The designer has complete flexibility to incorporate anticipated increases or decreases in traffic volume by aircraft type or take into account future aircraft types within the total forecast.

Figure 14 shows a plot of the typical output for the given aircraft mixture. The results shown are only for h_1 = 10 in., 20-year service life, and asphalt concrete tensile strain (fatigue) mode of distress. For those conditions, the maximum number of DC-8-63F strain repetitions due to the mix is approximately 130,000, and the maximum damage location occurs at about ±9.5 ft from the taxiway centerline.

Table 4 gives the summary version of the program output data. That portion summarizes the maximum strain repetitions and location of maximum damage, 4 levels of asphalt concrete thickness, and a 20-year service life. Similar results are also

Figure 12. Effect of various design parameters on ratio of principal tensile strains.

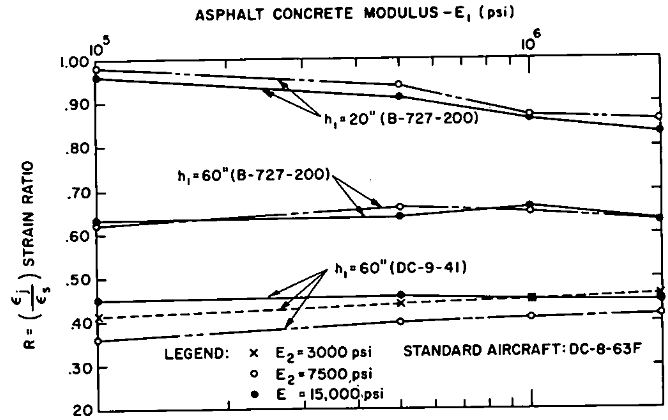


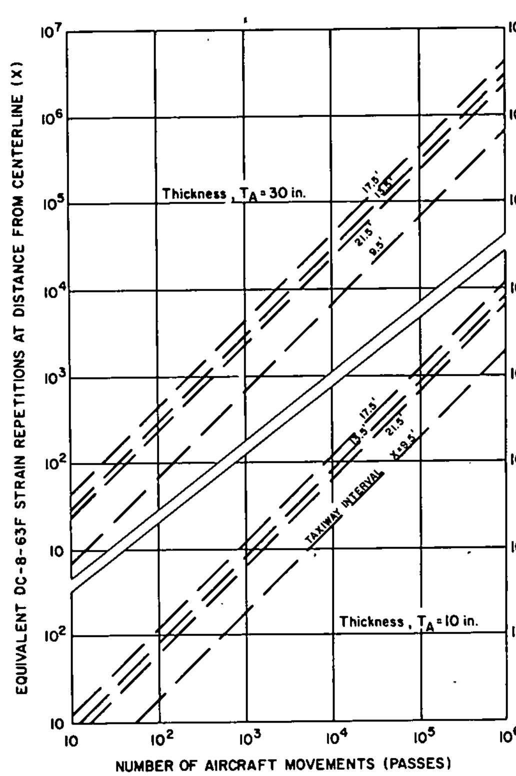
Table 2. F_{jh} values for various asphalt concrete pavement thicknesses.

Aircraft	10 In.	20 In.	30 In.	40 In.	50 In.
L-500	0.368	0.721	1.098	1.49	1.832
B-747-F	0.594	1.383	2.197	3.045	3.742
B-747	0.392	0.876	1.970	2.158	2.393
L-1011-8	1.692	3.843	6.234	8.542	10.863
DC-10-30	0.594	0.843	1.000	1.096	1.229
DC-10-10	0.700	0.736	0.824	0.752	0.796
L-1011-1	0.619	0.707	0.938	0.716	0.698
Concorde	0.820	1.432	1.665	2.335	2.652
DC-8-63F	1.000	1.000	1.000	1.000	1.000
B-707-320C	0.480	0.639	0.772	1.000	0.994
DC-8-61	0.635	0.626	0.652	0.638	0.602
B-707-120B	0.158	0.189	0.233	0.255	0.270
CV-990	0.277	0.446	0.547	0.606	0.698
B-720B	0.113	0.149	0.180	0.198	0.211
CV-880M	0.134	0.166	0.188	0.195	0.220
B-727-200	0.645	0.303	0.172	0.119	0.088
DC-9-41	0.264	0.076	0.037	0.022	0.015
B-737-200C	0.126	0.047	0.024	0.015	0.013
SE-210-6R	0.013	0.012	0.013	0.011	0.013
BAC-1-11-5	0.291	0.063	0.026	0.014	0.009
DC-9-15	0.084	0.026	0.011	0.007	0.005
VIS-810	0.069	0.015	0.006	0.003	0.002

Table 3. Taxiway passes for various service-life intervals.

Aircraft	0-5 Years	5-10 Years	10-15 Years	15-20 Years
DC-8-63F	2,000	3,000	5,000	8,000
DC-8-61	10,000	10,000	8,000	8,000
DC-9-41	9,500	12,000	13,500	16,000
DC-9-15	6,500	7,000	7,500	8,000
B-707-120B	7,500	6,000	5,000	4,000
B-720	2,000	2,000	2,000	2,000
B-727-200	40,000	50,000	65,000	80,000
B-737-200C	8,000	10,000	12,000	15,000
CV-880M	8,000	7,000	6,000	5,000
BAC-1-11	2,000	1,500	1,500	1,000

Figure 13. Typical aircraft fatigue equivalency diagram.



AIRCRAFT: L-1011-8

STRAIN CRITERION: A.C. TENSILE

Figure 14. Computer program results of aircraft traffic mix analysis.

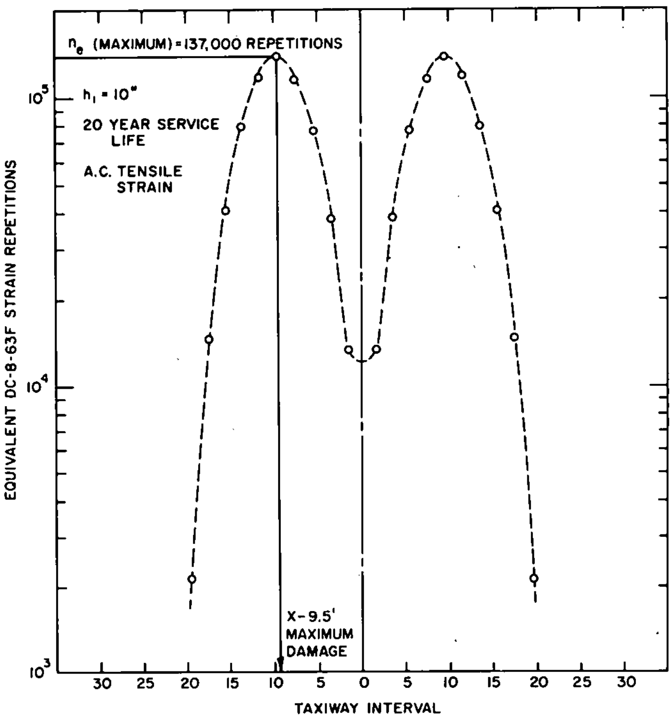


Table 4. Maximum equivalent DC-8-63F strain repetitions at various depths through year 20.

Analysis	Asphalt Concrete Thickness (in.)	Taxiway Interval (ft)	Max
Asphalt concrete tensile strain	10	9 to 10	137,103
	20	9 to 10	77,896
	30	9 to 10	57,101
	40	9 to 10	48,362
Subgrade vertical strain	10	9 to 10	134,099
	20	9 to 10	30,262
	30	9 to 10	20,162
	40	9 to 10	18,122

Figure 15. Predicted traffic value curve.

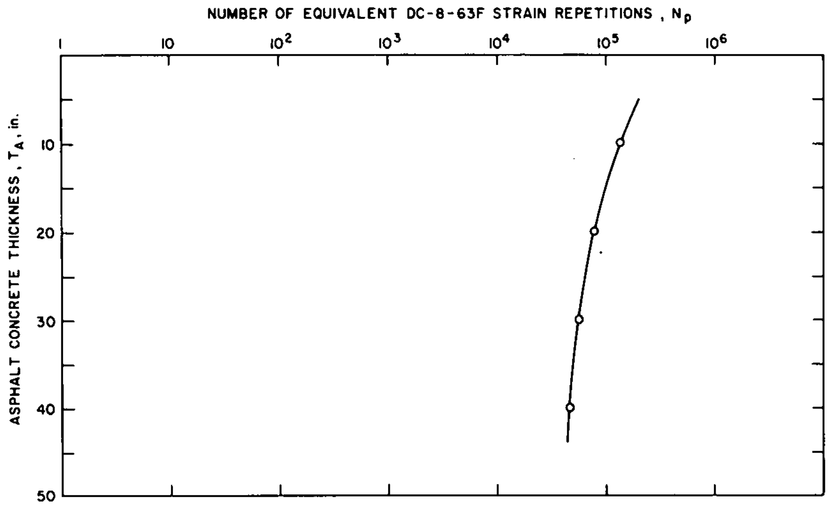
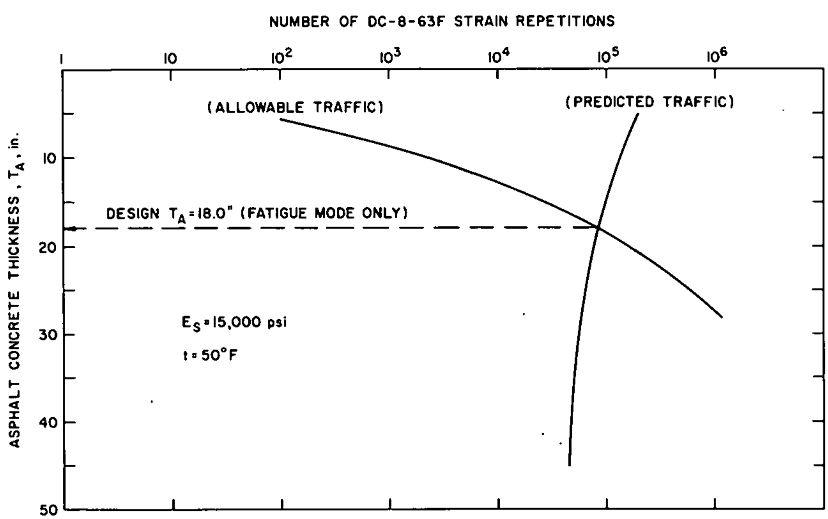


Figure 16. Solutions of full-depth asphalt concrete thickness requirements to satisfy fatigue distress mode.



generated for service lives of 5, 10, and 15 years for the mixture and allow the designer to use the traffic forecast for staged-construction purposes. The availability of the program allows a rapid method of analyzing even the most complex aircraft traffic mixtures. The final predicted traffic curve, for the fatigue mode of distress, for the data given in Table 6 is shown in Figure 15.

SUMMARY

The solution of the predicted traffic value curve allows the final design thickness requirement for the fatigue distress mode to be determined from the allowable traffic value results. The solution is made by a simultaneous graphical solution of both traffic value curves. Figure 16 shows the solution for the subgrade modulus and temperature data shown in Figure 10 and the 20-year design analysis for the aircraft mix given in Table 3.

For that problem, the full-depth thickness requirement for the fatigue mode of failure is 18.0 in. That thickness requirement only satisfied the fatigue criterion, and another separate analysis has to be made to determine the thickness requirements to satisfy the permanent deformation (subgrade shear failure) mode of distress. The final design thickness requirement would be the larger of those 2 values.

REFERENCES

1. Ahlvin, R. G., and Brown, D. N. Stress Repetitions in Pavement Design. Jour. of Aero-Space Transp. Div., Proc., ASCE, Oct. 1965.
2. ASCE Airfield Pavement Committee. Effect on Total Traffic of Moving Taxiway Center Line. ASCE Newsletter, Sept. 1969.
3. Barksdale, R. D. Compressive Stress Pulse Times in Flexible Pavements for Use in Dynamic Testing. Highway Research Record 345, 1971, pp. 32-44.
4. Deacon, J. A. Equivalent Passages of Aircraft With Respect to Fatigue Distress of Flexible Airfield Pavements. Proc., AAPT, 1971.
5. Dormon, G. M., and Metcalf, C. T. Design Curves for Flexible Pavements Based on Layered Theory. Highway Research Record 71, 1965, pp. 69-84.
6. Edwards, J. M., and Vakering, C. P. Structural Design of Asphalt Pavements for Heavy Aircraft. Shell Construction Service, 1971.
7. Heukelom, W., and Foster, C. R. Proc., ASCE, Vol. 86, 1960, pp. 1-28.
8. Kallas, B. F. Asphalt Pavement Temperatures. Highway Research Record 150, 1966, pp. 1-11.
9. Kingham, R. I. Failure Criteria Developed From AASHO Road Test Data. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972.
10. Marek, C. R., and Dempsey, B. J. A Model Utilizing Climatic Factors for Determining Stresses and Deflections in Flexible Pavement Systems. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972.
11. Full-Depth Asphalt Pavements for Air Carrier Airports, 2nd Ed. The Asphalt Institute, MS-11.
12. Turnbull, W. J., Foster, C. R., and Ahlvin, R. G. Design of Flexible Pavements Considering Mixed Loads and Traffic Volume. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Aug. 1962.
13. Study of Channelized Traffic. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Tech. Memo. 3-426, Feb. 1956.
14. Study of Lateral Distribution of Aircraft Traffic on Runway. U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Misc. Paper 4-369, Jan. 1960.
15. Witczak, M. W. Asphalt Concrete Overlay Requirements for Runway 18-36, Washington National Airport. The Asphalt Institute, Res. Rept. 72-4, Aug. 1972.
16. Witczak, M. W. Design Analysis: Full-Depth Asphalt Pavement for Dallas-Fort Worth Regional Airport. The Asphalt Institute, Res. Rept. 70-3, Nov. 1970.
17. Witczak, M. W. Design of Full-Depth Asphalt Airfield Pavements. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972.
18. Witczak, M. W. Prediction of Equivalent Damage Repetitions From Aircraft Traffic Mixtures for Full-Depth Asphalt Airfield Pavements. Proc., AAPT, 1973.

Elastic theory and 40 years of empirical flexible pavement design in Kentucky have been joined into the design system presented in this paper. A brief discussion is presented of the coupling mechanisms relating experience to theoretical analyses. An annotated design procedure is presented as a guide for pavement designers. Design nomographs account for a wide range of input parameters and permit the designer a wide choice of alternative thickness designs.

Pavement Design Schema

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The approach to a structural engineering problem is to resolve an equation of equilibrium and an equation of failure. The simplest equilibrium equations are found in elastic theory. The simplest failure equations are statements of phenomenological strengths. A rational design criterion for pavements must be compatible with all past experience and performance histories. In fact, collectively, those experiences are the best available equations of failure. Empirical design systems qualify abundantly in this way.

Many logic statements may be needed to transform empirical parameters into classical units and to bring experiences into conformity with strict mechanistic disciplines. When they are so transformed and anomalies are resolved, the predictive capabilities of the mechanistic theory stand confirmed; and the schema is claimed to be rational. Indeed, an enabling element in this venture was the Chevron computer program (1) to solve N-layered, elastic theory problems. The empirical resources were contained in a well-developed, experience-tested, equivalent wheel load (EWL)-California bearing ratio (CBR)-thickness design criterion or system (2, 3, 4).

From the mechanistic point of view, load-deflection relations outwardly portray the composite stiffness or rigidity of pavement systems. Contrary to general impressions, surface deflection is not a discrete, limiting parameter. Stresses and strains in the subgrade soil and in the extreme fibers of bituminous concrete layers constitute overriding, fundamental limits. Therefore, thickness design criteria cannot be based directly on deflection spectra.

It is historically evident that many pavements fail through fatigue and creep. In the fatigue domain, the state of strain and stress or both is computable from elastic theory. Obviously, it is necessary to resolve a suitable fatigue diagram. Customarily, fatigue diagrams are in terms of either controlled strains or controlled stresses. Creep alludes to the mechanism of rutting and is most easily handled in a separate analysis. In that instance, creep, or rutting, is handled empirically.

DEVELOPMENT OF DESIGN PROCEDURE

The controlling, empirical model in this instance was the 1958 Kentucky design curves shown in Figure 1 (4). It involved 3 parameters and 3 layers. By convention, the total thickness has been proportioned to be approximately ⅓ asphaltic concrete and ⅔ crushed rock base. Control points were selected for matching and balancing the elastic theory and fatigue analyses. Analysis of computer (Chevron program) results prevailed in the rightward portion of Figure 1, that is, to correct earlier errors in judgment in placing the design curves. Of course, the objective was to reconstitute those curves through theory (5, 6). Layer moduli and thicknesses were arrayed, and many solutions were obtained; numerous influence graphs were plotted. The necessary input assumptions are given below.

1. E_1 (modulus of elasticity of layer 1) ranged from 150,000 to 1,800,000 psi. The effective moduli of asphalt-bound layers depend on the pavement temperature and time of loading. Subgrade strains are critical when the asphaltic layer is warm and its modulus of elasticity is relatively low. On the other hand, strains in the asphaltic layer are critical at lower temperatures when its modulus is relatively high.
2. Poisson's ratio of layer 1 = 0.40. Dormon and Edwards (7) have reported that Poisson's ratio of such materials varies from 0.35 to 0.45.
3. E_2 (modulus of elasticity of layer 2) = F × CBR × 1,500, where F is found from curves shown in Figure 2 (5, 6, 8); note that F = 1 when $E_1 = E_2 = E_3$. Heukelom and Klomp (9) have shown that the effective elastic moduli E_2 of granular base courses tend to be related to the modulus of the underlying subgrade soil. The ratio of the base modulus to the subgrade modulus is a function of the thickness of the granular base, and in situ test results show that the range of that ratio is generally between 1.5 and 4.0. A value of 2.8 was selected in this study as being typical at a CBR of 7 (Fig. 2). Comparison of the 1958 Kentucky design curves and field data (4) indicated that this assumption was reasonable. It was further assumed that the ratio of E_2 to E_3 would be equal to one when $E_1 = E_2 = E_3$. The curves shown in Figure 2 were then obtained by assuming a straight-line relation on a log-log plot. A review of the literature (10, 11) indicated that curves shown in Figure 2 give reasonable values for good quality granular bases within a range of practical design situations (CBR < 20); and, therefore, they were used throughout the analysis here to relate the modulus of the granular base to the subgrade support values. E_2 values are a function of E_1 and E_3 only.
4. Poisson's ratio of layer 2 = 0.40. Again, Dormon and Edwards (7) have reported Poisson's ratio of 0.35 to 0.45.
5. E_3 (modulus of elasticity of layer 3) = CBR × 1,500. Conversion from laboratory soil strength values to theoretical moduli of subgrades was aided by Heukelom and Foster (12), who developed a relation suggesting the subgrade modulus (in psi) is approximately equal to the product of the CBR and 1,500. Heukelom and Klomp (9) also indicated that this relationship is an acceptable approximation for evaluating subgrade moduli and provides a simple and practical approach to this estimation, at least for CBR's up to about 20.
6. Poisson's ratio of layer 3 = 0.45. Dormon and Edwards (7) indicated Poisson's ratio for subgrade materials on this order.
7. Tire pressure = 80 psi. Many firms in Kentucky indicated that they operated their trucks with a tire pressure of 80 psi.

A summary of the derivation of the fatigue criterion follows.

1. Kentucky EWL's (equivalent 5,000-lb wheel loads) were transformed into EAL's (equivalent 18-kip axle loads) (5, 6) by EAL's = 2-directional Kentucky EWL's/32.
2. The criterion concerning limiting strains in the asphaltic concrete was based on interpretative analyses of other work (13). Van der Poel (14, 15) indicated that a safe limit for asphalt was in the order of 1×10^{-3} at 30 F. Because asphaltic concrete consists of approximately 10 percent binder by volume, that fixes the safe strain level of asphaltic concrete at 30 F in the order of magnitude of 1×10^{-4}. Others (7, 13, 16, 17) have established (by interpretative analyses of pavements and fatigue test data) that the

magnitude of asphalt strain ϵ_A to ensuring 1×10^6 repetitions at 50 F was 1.45×10^{-4}. Limiting values of strain (all at 50 F) as a function of number of repetitions N of the base load (18-kip axle load in EAL computations) as given by Dormon and Metcalf (17) can be represented by the equation $\log \epsilon_A = -3.84 - 0.199 (\log N - 6.0)$. Other fatigue curves representing other temperatures, i.e., other values for E_1, were derived from curves shown in Figures 3 and 4. The relations given by Kallas (18) between temperature and E_1 provided guidance at this stage.

Some investigators suggest a fatigue diagram of the load-log N type. Fatigue theorists (19, 20, 21) have suggested and shown in certain instances that a log load-log N plot is more realistic. Pell (20) suggested an equation of the form $N = K'(1/\epsilon_A)^n$, where n is the slope of the log ϵ_A-log N plot and K' is a constant. Pell (20), Deacon (19), and others have suggested that the value of n lies between 5.5 and 6.5 and is a function of the modulus of the asphaltic concrete. Pell's work further suggested that the family of curves relating log ϵ_A to log N for different E_1 values is parallel. The use of such a relation in this study produced such irrational results (as E_1 decreased, the total pavement thickness decreased) that an alternative relation was sought.

By plotting (to a log-log scale) the 18-kip tensile strain versus the tensile stress at the bottom of the asphaltic layer, we noted that for a given E_1 the curves depicting structural influences appeared to converge to a single point near a strain of 2×10^{-3} (Fig. 3). By extrapolating Dormon and Metcalf's data (17), represented by the equation given above, to a value of N = 1, we found the asphaltic tensile strain to be 2.24×10^{-3}. That strain was thus taken to be the limiting or critical asphaltic tensile strain for a single application of a 9-kip wheel load. By constructing lines tangent to the strain versus stress curves at a strain of 2.24×10^{-3}, we obtained modulus lines representing the limiting relations for asphaltic strain versus stress—independent of structural influences. The stress-strain ratios shown in Figure 3 are in terms of bulk moduli ($E_1 = 0.6K_1$, where K_1 is the bulk modulus).

For a total pavement thickness consisting of 33 percent asphaltic concrete thickness (with a modulus of 480 ksi, typical of pavements in Kentucky), it was observed that the tensile strain at the bottom of the bound layer for a CBR of 7 and total thickness of 23 in. (control pavement) was 1.490×10^{-4}. The traffic associated with that control point was 8×10^6 EAL's. In Figure 3, a line drawn perpendicular to the line for an asphaltic concrete modulus of 480 ksi, as determined above, at a strain of 1.490×10^{-4} intersected the other asphaltic moduli lines at strains that were assumed to be critical strains at 8×10^6 EAL's. Based on a straight-line variation between log ϵ_A and log N, the curves shown in Figure 4 were obtained as representing the critical asphaltic concrete strains.

The limiting asphaltic stress-strain curves shown in Figure 3 are shown again in Figure 5. For any given modulus of asphaltic concrete, the limiting strain for a single application of a catastrophic load $[EAL = N(1.25)^{P-18}]$, where P is the axle load in kips (5, 6), is taken to be 2.24×10^{-3}. As shown in Figure 4, another known point of limiting strain falls on the line perpendicular to the stress-strain curves for 8×10^6 repetitions. Based on a logarithmic scale between these 2 points, the lines of equal numbers of repetitions shown in Figure 5 are obtained. The limiting asphaltic concrete tensile strain for any combination of number of repetitions and modulus of the asphaltic concrete can be read from curves shown in Figure 5 and are the same as those shown in Figure 4. The curves shown in Figure 5 converge to a common strain value at N = 1. That is a unique feature in the development of the schema. The convergence allows stress to proportionalize according to modulus when a limiting catastrophic strain is respected, regardless of modulus.

3. It was observed from computations and analysis (5) that the vertical strain at the top of the subgrade ϵ_s for the control pavement (CBR 7, 23-in. total pavement thickness, i.e., 7.7 in. of asphaltic concrete and 15.3 in. of crushed stone base) was 2.400×10^{-4}. A review of other work (10, 17) also indicated that an ϵ_s of 2.400×10^{-4} for 8×10^6 18-kip axles would provide a high degree of assurance against rutting; that value was thus assigned to ϵ_{s9} at 8×10^6 repetitions and a wheel load of 9 kips. Analysis of elastic theory computations throughout a spectrum of pavement structures resulted in the curves shown in Figure 6 (5, 6). Figure 7 was then prepared and can be used to determine the limiting vertical strains at the top of the subgrade for various equivalent single wheel loads and thus for various values of accumulative EAL's.

Figure 1. 1958 Kentucky flexible pavement design curves.

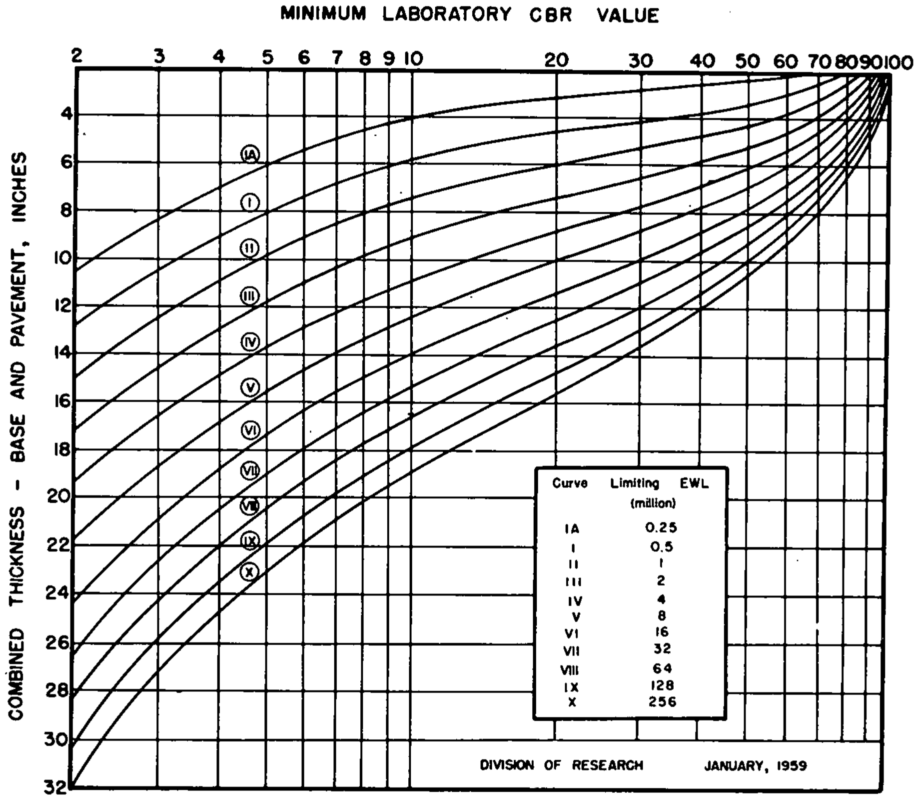


Figure 2. Relation of moduli of subgrade and moduli of granular base.

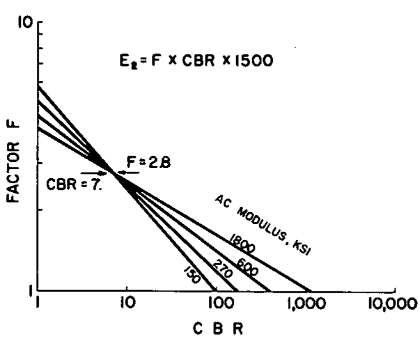


Figure 3. Asphaltic tensile strains-stresses for various structure CBR, and asphaltic concrete moduli of elasticity.

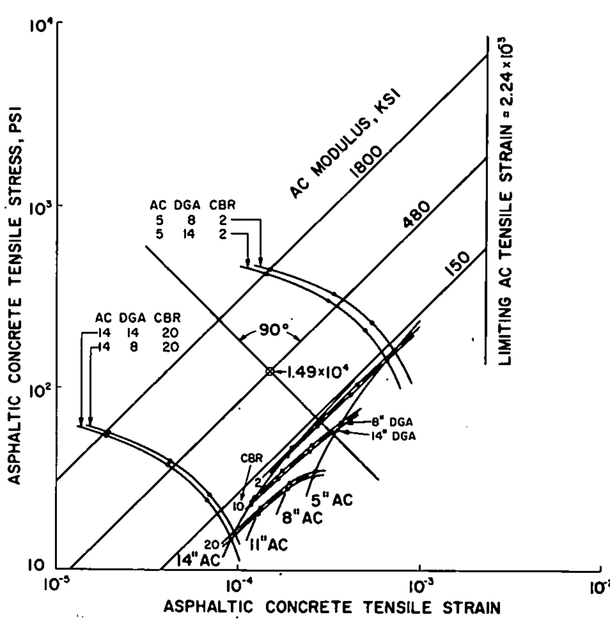


Figure 4. Asphaltic stress-strain curves showing application of strain-control criterion.

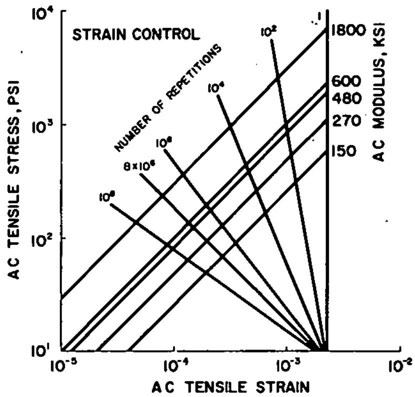


Figure 5. Limiting asphaltic concrete tensile strain as function of number of repetitions and asphaltic concrete moduli of elasticity.

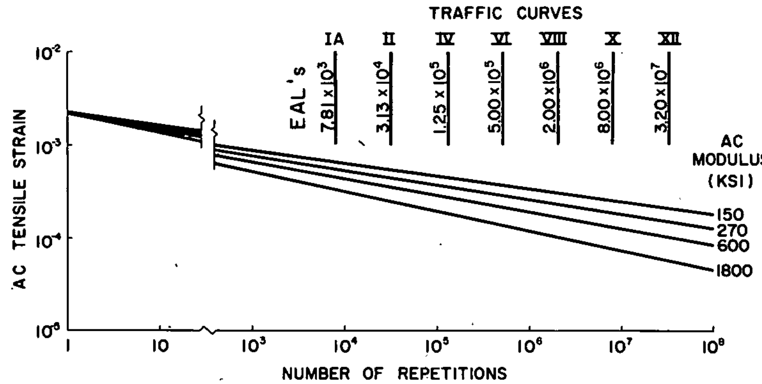


Figure 6. Ratio of subgrade strain to strain under 9-kip wheel load as function of equivalent, hypothetical wheel load.

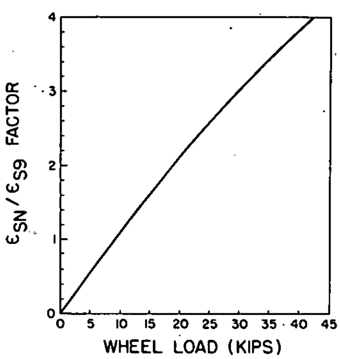
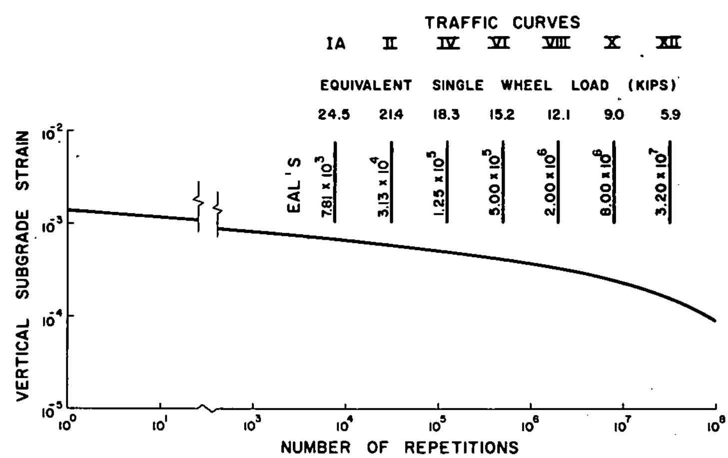


Figure 7. Limiting subgrade vertical compressive strain as function of number of repetitions and equivalent single axle load.



4. To complete the fatigue analysis required that results be plotted in terms of modulus values, layer thicknesses, and so on from influence graphs, satisfying limiting strains. That was done for the following proportions of T_1 and T_2: $T_1 = \frac{1}{3}$ T and $T_2 = \frac{2}{3}$ T, $T_1 = \frac{1}{2}$ T and $T_2 = \frac{1}{2}$ T, $T_1 = \frac{3}{4}$ T and $T_2 = \frac{1}{4}$ T, and T_1 = T and T_2 = 0. T_1 = thickness of layer 1, T_2 = thickness of layer 2, and T = total pavement thickness. Coaxial graphs, shown in Figures 8, 9, and 10, were drawn to permit continuous interpolations.

DESIGN PROCEDURE

Design Period (and Design Life)

The design life is the time period of useful performance and is normally considered to be 20 years. Pavements may be designed for an ultimate 20-year life but be constructed in stages. Low-class roads may be designed in stages or merely designed for a proportionately shorter life. Usually it will not be practical to design pavements for low-class roads to last 20 years. Economic analysis or limitations of funds may dictate the design period.

Traffic Volume Information

Normally, traffic volumes are forecast in connection with needs studies and in the planning stages for all new routes and for major improvements of existing routes. Whereas anticipated traffic volume is an important consideration in geometric design, the composition of the traffic in terms of axle weights, classifications, and lane distributions is essential to the structural design of the pavement. Traffic volumes used for EAL computations should, therefore, be reconciled with other planning forecasts of traffic. Historically, actual growths of traffic have exceeded the forecasts in the majority of cases. Overriding predictions of traffic volumes may be admissible for purposes of EAL estimates when properly substantiated. Moreover, the design life of the pavement may differ from the geometric design period.

If only the beginning and twentieth-year AADT is furnished, it may become necessary to request a listing of AADT estimated for each calendar year; otherwise, a normal growth curve must be assumed. In the absence of specific guiding information, a constant yearly increase factor may suffice, typified by the compound interest equation

$$A = P(1 + i)^n$$

where

A = AADT in the nth year,
P = the beginning AADT,
i = yearly growth factor, and
n = number of years from the beginning.

Thus, the AADT for each year may be calculated and then summed through n years; or an "effective" AADT may be calculated as (P + A)/2, which, when multiplied by the number of years, yields a cursory estimate of the total design-life traffic.

Design EAL'S

Heretofore, the Kentucky design system was based on EWL's. The present system is based on EAL's. That transformation was made for the sake of unifying design practices and standardizing definition of design terms. EAL's are defined here as the number of equivalent 18-kip axle loads (22).

Basically, the computation of EAL's involves, first, forecasting the total number of vehicles expected on the road during its design life and, second, multiplying by factors to convert total traffic to EAL's (23). Of course, that is obviously an extreme simplification. More ideally, the yearly increments of EAL's could be calculated and summed; that approach would permit consideration of anticipated changes in legal weight

Figure 8. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 33 percent of total pavement thickness.

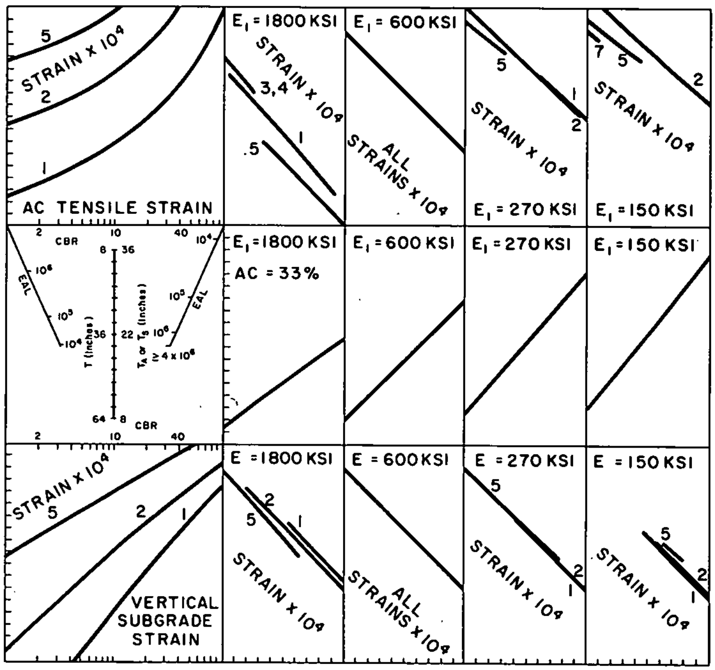
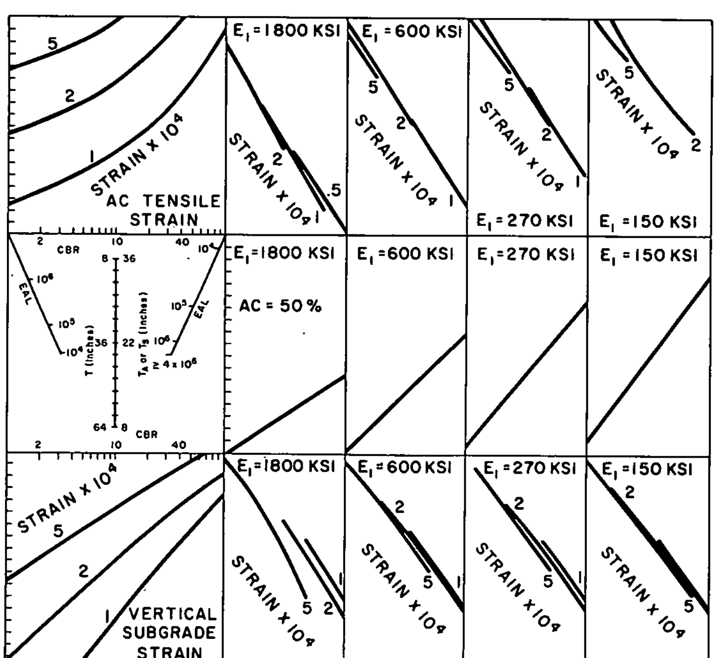


Figure 9. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 50 percent of total pavement thickness.



limits, changes in style of cargo haulers, and changes in routing. If a design life of fewer than 20 years is to be considered or if staged design and construction is foreseen, the EAL value for the respective design period is determined.

The EAL's so determined are gross, 2-directional values that must be reduced to 1-directional values. When more than 2 lanes in each direction are involved, additional factors appropriating EAL's among the lanes will be necessary. No guiding values may be cited, but such values should be available from the planning study report. The necessity of those factors is apparent: It is customary to design all lanes like the most critical one; adjacent lanes of different thicknesses might result in complicating construction procedures. The validity of such a line of argument, however, may be subject to question in the future (24).

Design CBR

CBR test values (3) reflect the supporting strength of the subgrade. Moreover, the test procedure intentionally conditions the soil (by soaking) to reflect its least or minimum supporting strength; that is presumed to be representative of the soil strength during sustained wet seasons when the ground is saturated or nearly so. At other times, the soil may be much stronger; and pavements thereon would be capable then of withstanding heavier loads. If pavements were not designed for the minimum capabilities of the foundation soil, it might be necessary to impose further restrictions seasonally with respect to single axle loads in order to prevent premature and catastrophic failures. However, a pavement should be designed so that it will perform adequately throughout the design period when seasonal variations are considered. To the extent that such performance is represented, the empirical curves shown in Figure 1(and thus the corresponding empirical expressions of failure criterion) represent such designs.

The CBR value does not ensure immunity against frost heave even though it may have a compensating effect in the design of the pavement structure. Greater pavement depths are required for low-CBR soils than for high-CBR soils, and it is usually the low-CBR soils that are more sensitive to frost. A high type of pavement is normally of sufficient thickness that the supporting soil lies below the freezing line (in Kentucky). However, because of the thermal properties of the constituent materials of the pavement, frost penetration in the pavement may be greater than in the adjacent soil mass. For thinner pavements, the supporting soil is well within the frost zone; therefore, the pavement structure providing the greatest template depth is preferred. Pavements less than 6 in. in thickness or having less than 4 in. of asphaltic concrete should be regarded dubiously from this point of view. It is recommended that soil having a CBR of less than 2 be considered ineligible and unsuitable for use as pavement foundation.

Soil surveys may indicate wide variations in CBR along the length of a specific route. It is presumed that adequate pavement thicknesses will be provided throughout the project. The designer must, therefore, consider the contiguity of the soils and perhaps sectionalize the project according to minimum CBR. The designer must respect all minimums, or else some sections of pavement will be underdesigned; overdesigns must be admitted as a natural consequence therefrom. The designer is privileged to decide whether to require an intervening low-CBR section to be upgraded to the same quality as that of abutting high-CBR sections or make a separate design for the low-CBR section. Of course, the designer should consider the relative economics of the 2 alternatives, but he may also consider continuity and uniformity of pavement section and construction control as pertinent factors. Usually it will be found impractical to vary the design thickness within short distances.

Asphaltic Concrete Modulus of Elasticity

Generally, design systems do not account for the possible range of values of the modulus of elasticity of bituminous concrete. That has generally proved to be more than adequate because such design systems have been applied to rather limited situations in which the stiffness characterization of bituminous mixtures actually used in practice falls within a very limited range. The effective moduli of asphalt-bound layers depend on the pavement temperature and time of loading. As design systems begin to take into account to greater degrees the range of pavement temperatures and times of loading, the modulus of the bituminous concrete mixture becomes more and more significant.

Initial and preliminary analysis of the performance of Kentucky flexible pavements (thickness being ⅓ asphaltic concrete and ⅔ crushed stone base) in comparison with theoretical computations indicates empirically that the bituminous concretes used in Kentucky typically have an apparent modulus of elasticity of about 480,000 psi; that corresponds to the modulus at 64 F (the mean annual pavement temperature) obtained from an independent correlation between modulus and average pavement temperature. Weighting distributions of pavement temperature of more than 64 F for various thicknesses of asphaltic concrete suggest that 76 F might be considered an equivalent "design" temperature for full-depth asphaltic concrete pavements.

Designs with lesser proportions of the total thickness being asphaltic concrete might be expected to be less sensitive to rutting of the asphaltic concrete than full-depth designs. The reduced susceptibility might be considered as an increase in the effective modulus of elasticity of the asphaltic concrete. Correlating the mean pavement temperature with the modulus of elasticity of the asphaltic concrete according to Southgate and Deen (25) makes it possible to determine and plot (Fig. 11) the moduli corresponding to 64 F (thickness being ⅓ asphaltic concrete) and 76 F (full-depth asphaltic concrete). Based on a straight-line relation, the change in asphaltic concrete modulus as the temperature sensitivity to rutting varies is described as shown in Figure 11. Designs obtained by the use of modulus values shown in Figure 11 would surely perform at least equal to current designs (employing usual proportions of dense-graded aggregate base and asphaltic concrete surface courses). Other more refined weightings should be regarded as admissible.

Alternative Pavement Thicknesses

1. If the design EAL is known, the limiting subgrade strain can be determined from curves shown in Figure 7. Likewise, Figure 5 shows the limiting asphalt tensile strain values. If a design is desired for an asphaltic concrete with a modulus other than the 4 specifically shown in Figures 8, 9, and 10, it will be necessary to know the limiting asphaltic concrete strain for each of the 4 modulus values so that interpolations can be made later.
2. Enter the top portion (for asphaltic strain control) of Figure 8 at the design CBR. Draw a line vertically to limiting strain values (from Fig. 5) for each E_1; mark each point (Fig. 12).
3. Draw horizontal lines from each of the points obtained above to the respective E_1 modulus quadrants, and mark the point at the appropriate strain values.
4. From those points, draw lines vertically, and mark points on the turning lines.
5. From those points, draw lines horizontally, and read T_A values for each E_1 modulus on the thickness scale.
6. Repeat step 2 but use the lower portion (for subgrade strain control) of Figure 8. Only one value of limiting subgrade strain is given for a fixed value of repetitions and is independent of E_1 moduli.
7. Draw a horizontal line to the right through all 4 quadrants and locate the strain value in each quadrant.
8. Repeat steps 4 and 5 to obtain values of T_S for each E_1 modulus.
9. Plot each design total thickness from steps 5 and 8 (arithmetic scale) versus log E_1 modulus, and fit a smooth curve to the points as shown in Figure 13.
10. Repeat steps 1 through 8 and use Figures 9 and 10.
11. From Figure 13, read the total thickness T_A for each ratio of thickness of asphaltic concrete to total thickness, and plot the resulting total thickness values (arithmetic scale) versus log of percentage asphaltic concrete thickness as shown in Figure 14. Repeat this step and use T_S from Figure 13.
12. Select from Figure 14 the final design total thickness values for T_A and T_S for the desired ratio of asphaltic concrete thickness to total thickness.

Figure 10. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 100 percent of total pavement thickness.

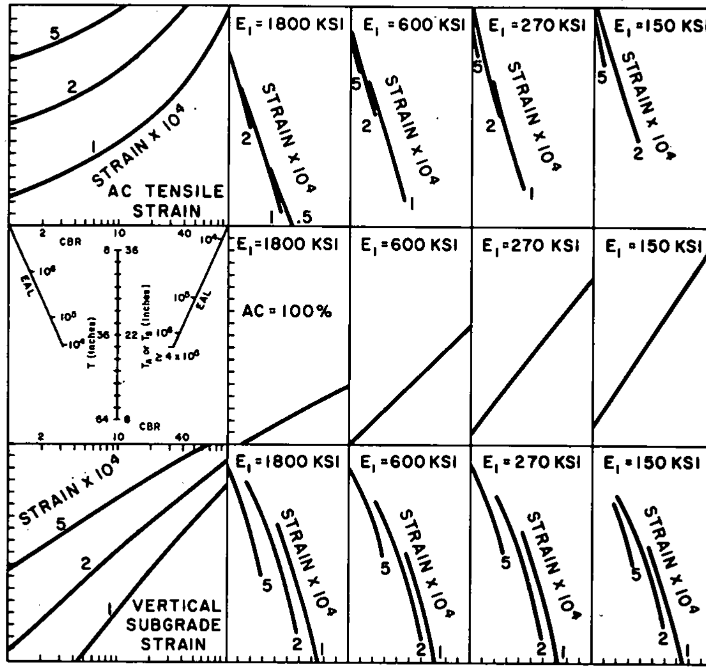


Figure 11. Weighting of asphaltic concrete modulus for ratio of thickness of asphaltic concrete to total pavement thickness.

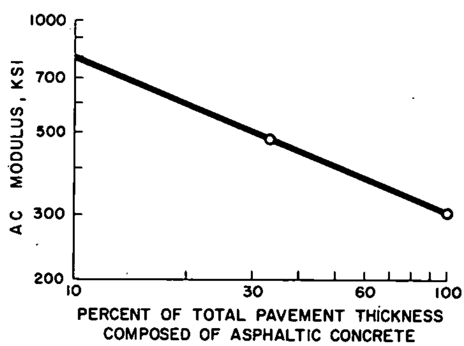


Figure 12. Illustration of use of Figure 8.

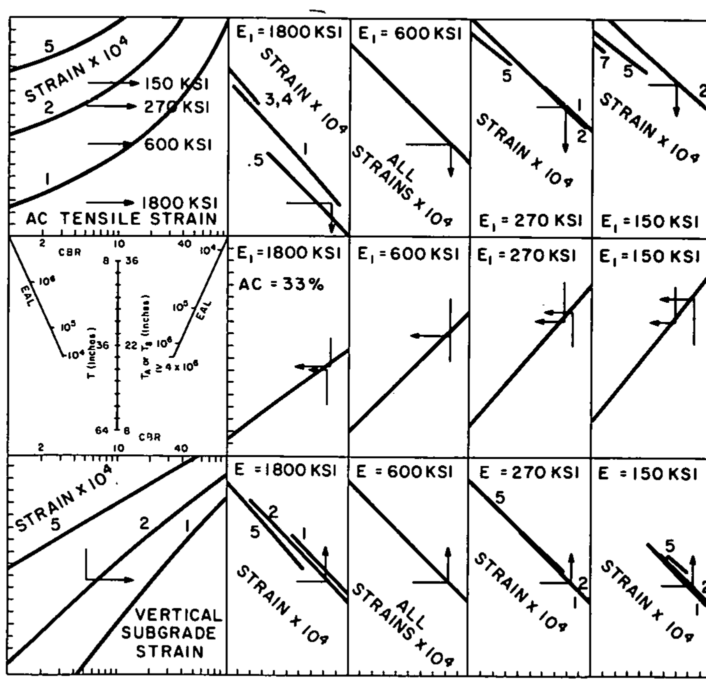


Figure 13. Total pavement thickness, T_A and T_S, as function of asphaltic concrete modulus.

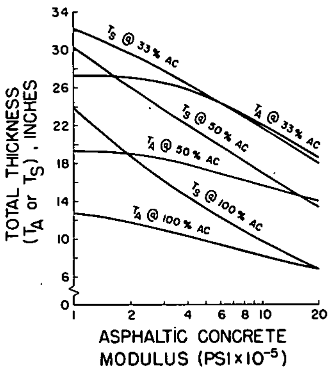


Figure 14. Total pavement thickness, T_A and T_S, as function of ratio of thickness of asphaltic concrete to total thickness.

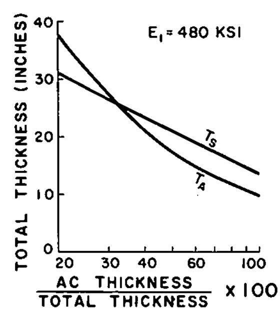


Figure 15. Nomograph to adjust design thicknesses for rutting.

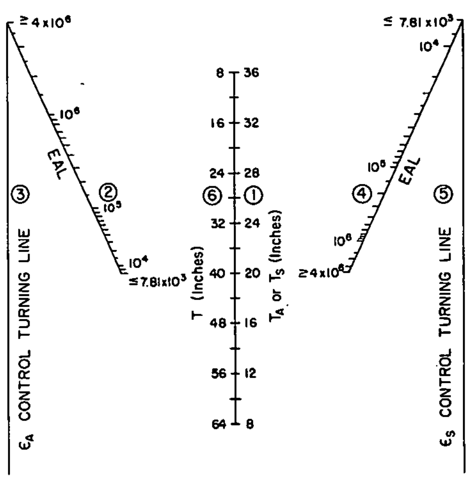
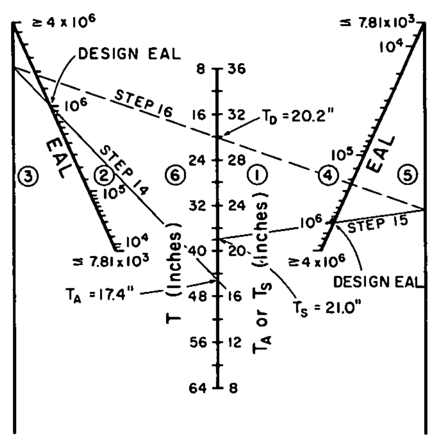


Figure 16. Illustration of use of Figure 15.



13. If the design EAL is 4×10^6 or greater, the design total thickness for each E_1 modulus is the greater of T_A and T_S. If the design EAL is 7.81×10^3 or less, the total thickness design is T_A.

Rutting of Subgrade

Whereas the respective design curves provide equal assurances against rutting throughout all ranges of EAL's, greater rutting is tacitly and progressively admissible in some inverse relation to EAL's. It has been presupposed that no additional rutting should be allowed in pavements having design EAL's equal to or greater than 4×10^6. On the other hand, it seemed that a pavement having a design EAL equal to or less than 7.81×10^3 might be allowed to rut in a completely uncontrolled manner. Weighting the intervening curves in relation to EAL's permitted construction of a nomograph (Fig. 15) for those designs where rutting criteria control. It is suggested that this weighting be respected in an advisory way. It may be violated permissively in either direction, provided the fatigue limit of the asphaltic concrete layer is respected.

Figure 15 is used to adjust for rutting when the design EAL is more than 7.81×10^3 and less than 4×10^6. The final design thickness adjusted for rutting is obtained from the following procedure:

14. For the desired ratio of asphaltic concrete thickness to total thickness shown in Figure 14, read the total thickness (T_A for asphaltic concrete strain control), and mark on scale 1 in Figure 16. Draw a straight line from T_A on scale 1 through the design EAL value on scale 2, and mark the intersection point on line 3.
15. For the desired ratio of asphaltic concrete thickness to total thickness shown in Figure 14, read the total thickness (T_S for subgrade strain control), and mark on scale 1. Draw a straight line from T_S on scale 1 through the design EAL value on scale 4, and mark the intersection point on line 5.
16. Connect the intersection points on lines 3 and 5 by a straight line, and read the final adjusted design thickness on scale 6.

CONCLUDING REMARKS

To determine pavement thicknesses from the nomographs similar to those shown in Figures 8, 9, and 10, one must know design EAL's, CBR of the subgrade soil, and modulus of elasticity of the asphaltic concrete. Such a set of nomographs permits selection of pavement structures employing alternative proportions of bituminous concrete and crushed stone base. Total thickness varies according to the proportion chosen. However, the choice may not be made arbitrarily or trivially. It is implicitly intended that the final selection also be based on additional engineering considerations such as estimates of comparative construction costs, compatibility of cross-sectional template and shoulder designs, uniformity of design practices, highway system classifications, engineering precedence, and utilization of indigenous resources. Designs based on 33 percent and 67 percent proportions of bituminous concrete (asphaltic concrete modulus of 480 ksi) and crushed rock base respectively conform with the department's current design chart, representing current, conventional, or precedential design. The nomographs (Figs. 8, 9, and 10) represent theoretical extensions of conventional designs and, from a theoretical standpoint, provide equally competent structures.

REFERENCES

1. Michelow, J. Analysis of Stresses and Displacement in an N-Layered Elastic System Under a Load Uniformly Distributed on a Circular Area. Unpubl., Sept. 24, 1963.
2. Baker, R. F., and Drake, W. B. Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements. HRB Proc., Vol. 28, 1948, pp. 60-77.

3. Baker, R. F., and Drake, W. B. Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements. Eng. Exp. Sta., Univ. of Kentucky, Bull. 13, Sept. 1949.
4. Drake, W. B., and Havens, J. H. Re-Evaluation of the Kentucky Flexible Pavement Design Criterion. HRB Bull. 233, 1959, pp. 33-54.
5. Deen, R. C., Southgate, H. F., and Havens, J. H. Structural Analysis of Bituminous Concrete Pavements. Highway Research Record 404, 1972, pp. 22-35.
6. Southgate, H. F., Deen, R. C., and Havens, J. H. Rational Analysis of Kentucky Flexible Pavement Design Criterion. Div. of Research, Kentucky Dept. of Highways, 1968.
7. Dormon, G. M., and Edwards, J. M. Developments in the Application in Practice of a Fundamental Procedure for the Design of Flexible Pavements. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Univ. of Michigan, 1968.
8. Ahlvin, R. G., and Chou, Y. T. Discussion of paper by Deen, R. C., Southgate, H. F., and Havens, J. F., Structural Analysis of Bituminous Concrete Pavements. Highway Research Record 404, 1972, pp. 32-35.
9. Heukelom, W., and Klomp, A. J. G. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Univ. of Michigan, 1962.
10. Proceedings, 2nd Int. Conf. on Struct. Des. of Asphalt Pavements. Univ. of Michigan, 1968.
11. Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K. Prediction of Flexible Pavement Deflections From Laboratory Repeated-Load Tests. NCHRP Rept. 35, 1967.
12. Heukelom, W., and Foster C. R. Dynamic Testing of Pavements. Jour. Struct. Div., ASCE, ST1, Feb. 1960.
13. Mitchell, J. K., and Shen, C. K. Soil-Cement Properties Determined by Repeated Loading in Relation to Bases for Flexible Pavements. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Univ. of Michigan, 1968.
14. Van der Poel, C. Road Asphalt. In Building Materials (Reiner, M., ed.), Interscience Publishers, 1954.
15. Van der Poel, C. Time and Temperature Effects on the Deflection of Asphaltic Bitumens and Bitumen-Mineral Mixtures. Jour. Soc. Plastics Eng., Vol. 11, No. 7, Sept. 1955.
16. Lettier, J. A., and Metcalf, C. T. Application of Design Calculations to "Black Base" Pavements. Proc., AAPT, Vol. 33, 1964.
17. Dormon, G. M., and Metcalf, C. T. Design Curves for Flexible Pavements Based on Layered System Theory. Highway Research Record 71, 1965, pp. 69-84.
18. Kallas, B. F. Asphaltic Pavement Temperatures. Highway Research Record 150, 1966, pp. 1-11.
19. Deacon, J. A. Fatigue of Asphalt Concrete. Univ. of California, Berkeley, DEng thesis, 1965.
20. Pell, P. S. Fatigue of Asphalt Pavement Mixes. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Univ. of Michigan, 1968.
21. Kasianchuk, D. A. Fatigue Considerations in the Design of Asphalt Concrete Pavements. Univ. of California, Berkeley, PhD dissertation, 1968.
22. Committee on Design. AASHO Interim Guide for the Design of Flexible Pavement Structures. AASHO, Oct. 12, 1961.
23. Deacon, J. A., and Deen, R. C. Equivalent Axle Loads for Pavement Design. Highway Research Record 291, 1969, pp. 133-143.
24. A Guide to the Structural Design of Pavements for New Roads. Gt. Brit. Road Research Laboratory, Road Note 29, 1970.
25. Southgate, H. F., and Deen, R. C. Temperature Distribution Within Asphalt Pavements and Its Relationship to Pavement Deflection. Highway Research Record 291, 1969, pp. 116-131.

This paper deals with an application of the principles and concepts of fracture mechanics to the problem of cracking of flexible pavements under repeated loading. A brief discussion of the relevant principles of fracture mechanics is presented, and a summary is given of the theoretical and experimental work done at Ohio State University. The advantages and disadvantages of the method are discussed, and future developments are outlined.

Mechanistic Approach to the Solution of Cracking in Pavements

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The term "fatigue" is always associated with damage or deterioration under repetitive loading that eventually leads to cracking and sometimes catastrophic failure of the structural component. It implies a process of localized progressive structural change occurring in a material subjected to fluctuating stress that generally results in the lowering of the resistance of the material to subsequent stressing.

Fatigue is now recognized as a phenomenon of a highly complex nature, and it is generally accepted that no single theory can deal with all the relevant aspects of the problem. Ultimately many disciplines must be drawn together to develop a unified theory.

Existing theories tend to tackle the problem from only 1 of 3 points of view: statistical mechanics, microstructural, and continuum mechanics.

The statistical mechanics approach considers the problem on the basis of the kinetic concept of the mechanism of fracture that involves the breakage and reformation of atomic bonds by stress and thermal fluctuations, the accumulation of rupture bonds resulting in loss of stability, and eventually breakdown. That approach leads to quantitative results but suffers from the lack of consideration of the mechanics of the microstructures and the geometrical and boundary effects.

Microstructural theories describe the mechanism of crack initiation and growth. The type of microstructure of the material, crystalline or amorphous, and the loading conditions are considered. They tend to be qualitative because geometrical and boundary conditions are ignored.

The continuum mechanics approach is the most powerful. It considers the localized nature of the problem, the geometrical and boundary conditions, and conforms as closely as possible to the microstructural theories that are known to explain correctly the mechanism of fatigue.

The phenomenon of fatigue failure is associated with the concept of "damage" or those material changes that lead to formation of macroscopic cracks and subsequent structural instability. The occurrence of fatigue failure is a result of 2 separate processes:

damage initiation and damage growth. The occurrence of those 2 processes in a material system results in a gradual weakening of the structural components. However, the failure state is not reached until the damage approaches a critical level. In short, damage initiation and growth are necessary but not sufficient conditions for the occurrence of fatigue failure. In fact, damage growth in a material body can be arrested during the course of repeated loading before reaching the threshold of instability. The arrest of the damage can occur because either the applied load cannot furnish sufficient energy required for growth or other changes in the material body and boundary conditions alter the state of stress distribution in the structural component.

The processes of crack initiation and growth differ among various materials. Because of the presence of inherent flaws, it is reasonable to expect a crack to initiate at the first few cycles of load application (1) in certain alloys, plastics, polymers, and heterogeneous compositions such as asphaltic materials. The statistical distribution of such internal discontinuities can, in fact, account for statistical variations in the fatigue life.

The process of crack growth has been discussed by various theories, but fundamentally it is related to the deformation occurring at the tip of discontinuities and is associated with the energy balance in those regions. The work of external forces in the regions of discontinuity is divided into stored elastic energy, the energy required for irreversible changes in the material body as viscous or plastic flow, and the surface energy required to form a crack. The rate of crack growth then depends entirely on the energy balance, and the path it follows is governed by the minimum energy requirement.

During the cyclic deformation process, the tip of the zone of discontinuity blunts and resharpens, resulting in crack growth through the body (Fig. 1). That process continues until a crack of critical size has been reached, and the induced state of stress results in structural instability or terminal event of fracture.

Although the crack growth is discontinuous, it is assumed to be continuous to justify the use of the continuum mechanics approach. The problem of the initiation of crack growth and fracture belongs to the domain of fracture mechanics.

The development of fracture mechanics followed from Griffith's classical theory for brittle materials. Subsequently, Irwin and Kies proposed that a modified Griffith theory could be employed widely in fracture-strength analysis in the presence of substantial amounts of plastic strain as long as fracture occurred in advance of general yielding.

Modern fracture mechanics owes its development to Irwin, who proposed the concept of the stress-intensity factor. In 1957 he observed that all crack behavior could be classified into 3 distinct modes according to whether the resulting displacements contribute to the opening (mode 1), in-place sliding (mode 2), or tearing (mode 3), or modes of relative displacement of the crack surfaces (Fig. 2). The 3 modes are necessary and sufficient to describe all the possible modes of crack behavior in the most general state of elastic stress.

It follows naturally that each of the crack movements is associated with a stress field in the immediate vicinity of the crack tip. The distribution of stress in the vicinity of the crack tip is basically a problem in the mathematical theory of elasticity in which it can be shown that all crack-tip stress fields exhibit inverse square root singularities. Thus, for small-scale yielding (i.e., for a small plastic zone as shown in Fig. 1), the stresses in the vicinity of the crack tip can be expressed as follows for open mode 1:

$$\begin{pmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{xy} \end{pmatrix} = \frac{K_I}{\sqrt{2\pi r}} \cos\frac{\theta}{2} \begin{pmatrix} 1 - \sin\frac{\theta}{2}\sin\frac{3\theta}{2} \\ 1 + \sin\frac{\theta}{2}\sin\frac{3\theta}{2} \\ \sin\frac{\theta}{2}\cos\frac{3\theta}{2} \end{pmatrix}$$

There are similar expressions for the other 2 modes involving K_{II} and K_{III}.

The parameters K_I, K_{II}, and K_{III} are called the stress-intensity factors and clearly govern the magnitude of the local stresses in the vicinity of the crack tip.

The validity of the stress field is confined to an annular zone around the leading edge of the crack. The zone lies beyond the zone of plastic and nonlinear strains but does not extend beyond distances from the crack tip smaller than the crack and specimen dimensions.

The utility of the elastic stress field analysis lies in the similarity of the near crack-tip stress distributions for all configurations with the same stress-intensity factor; i.e., 2 bodies with cracks that are of different size and have different manners of load application but are otherwise identical will have identical near crack-tip deformation fields if the stress-intensity factors are equal. Or, in the words of Irwin, "The point of view so far represented is that of an imaginary small observer looking outward from the crack edge plastic strain zone, and [being] unable to distinguish whether an increase in the surrounding stresses arises from an increase in the applied load or from an increase in the size of the crack" (1).

The stress-intensity factor K has also been shown to be related to the Griffith strain energy release rate G as follows:

$$K^2 = \frac{GE}{1 - \nu^2}$$

for the plane strain, and

$$K^2 = GE$$

for the plane stress. $G = \delta W/\delta c$, the strain energy release rate, may also be considered as the crack extension force. Thus, K is seen to be a powerful parameter for it not only governs the magnitude of the stress field in the vicinity of the crack in accordance with the load, size of crack, and geometrical and boundary conditions but also is proportional to the force tending to cause crack extension.

It was not surprising, therefore, that in 1957 Irwin wrote that a substantial fraction of the mysteries associated with crack extension might be eliminated if some estimates of the stress conditions near the loading edge of the crack were made in terms of the stress-intensity factor.

However, it was not until 1961 that Paris, Gomez, and Anderson (2) first introduced the application of the stress-intensity factor to fatigue crack propagation rates. In 1963, Paris and Erdogan (3) found from experimental data that the crack propagation rate, dc/dN, was proportional to the fourth power of ΔK for a number of materials. This law of crack growth is expressed as $dc/dN = AK^n$, where A is a material constant and $n = 4$.

Later, the fourth-power relation was justified by consideration of the energy absorption within the entire plastic zone ahead of the crack tip (4). Rice (5) also derived Paris and Erdogan's expression by a rigid plastic model, which assumes plastic deformation is limited to a strip of material ahead of the crack tip.

It is clear from the discussions given above that the stress-intensity factor is the dominant parameter controlling the crack growth in a pavement, and therefore it is important to be able to determine its value, both theoretically and experimentally, for all modes of cracking in pavements as well as 2-dimensional simplifications used for simulation in laboratory experiments. The available theoretical solutions and experimental methodology are discussed in the following section.

DETERMINATION OF THE STRESS-INTENSITY FACTOR

To determine the stress-intensity factor K for a given crack size and specimen geometry, analytical methods have been developed for various boundary conditions.

The K-value for a simply supported beam with a central load was solved by Winne and Wundt and by Gross and Srawley (6). The results of their analysis are shown in

Figure 3. Finite element method has been used to develop a similar nondimensionalized relation among stress-intensity factor K, load, and beam geometries.

The K-value for a beam supported on an elastic foundation with a central load was obtained by the boundary collocation method. The solution is given in another paper (7). Similarly, a finite element program has been developed to calculate K for any crack size.

The solution for the stresses in a slab with a semi-infinite crack supported on an elastic foundation was obtained by Williams, Ang, and Folias (8). From that solution, the K-values under a moving load were obtained as given elsewhere (7).

The stress-intensity factor K for any type of loading, crack pattern, and geometry can be determined experimentally by a very simple procedure. That is done by measuring the change in the deflection as the cracks grow and by applying the formula

$$K^2 = \frac{P^2E}{2(1 - \nu^2)} \frac{\delta L}{\delta c}$$

where

P = load,
E = Young's modulus,
L = compliance, or inverse slope of the load-deflection diagram, and
c = crack length.

EXPERIMENTAL CHARACTERIZATION OF PAVEMENT SYSTEMS

In the research work carried out at the Ohio State University, the applicability of the theory to asphaltic materials has been examined in the light of 2 very important assumptions made in the theoretical concepts:

1. The material must be homogeneous, isotropic, and essentially elastic-plastic, and
2. The size of the plastic zone at the tip of the crack must be small in comparison to the crack and specimen dimensions.

The first assumption is one that is generally accepted for asphaltic materials. With regard to the second assumption, the size of the plastic zone r_y can be calculated from the formula

$$r_y = \frac{1}{2\pi}\left(\frac{K_1}{\sigma_y}\right)^2$$

where σ_y = yield stress in tension.

Using that estimate of the size of the plastic zone, Srawley and Roberts (9) established criteria for the crack length, width, and depth of beam to ensure plane strain conditions and the applicability of linear elastic fracture mechanics. The criteria state that both the crack depth and width of the beam should exceed

$$2.5\left(\frac{K_1}{\sigma_y}\right)^2$$

For a pavement, the theory is applicable if the thickness of the asphaltic layer is greater than $1.25\ (K_1/\sigma_y)^2$. For typical highway mixes, that criterion is satisfied even for pavement layers smaller than 1 in. and for the heaviest loads; therefore, from a practical point of view, we may say that linear elastic fracture mechanics is always applicable.

The results of experiments conducted at Ohio State University were reported in other papers (10, 11, 12, 13). The following salient points were made in those papers.

1. Tests on simply supported beams of sand asphalt beams tested at 23 F (10) showed that the rate of crack propagation dc/dN correlated well with the stress-intensity factor K in accordance with Paris' law

$$\frac{dc}{dN} = AK^4$$

where A is a material property.

2. The beams failed when the crack reached the critical crack length c_f corresponding to the critical stress-intensity factor K_{IC}. K_{IC} is the failure criterion for both monotonic fracture and fatigue and is a material property.

3. The fatigue life N_f of the beam may then be expressed as

$$N_f = \int_{c_o}^{c_f} \frac{1}{AK^4} dc$$

where c_o is the starter flaw. The starter flaw is a material constant but is subject to statistical variation and is believed to be principally responsible for the statistical variation of fatigue life.

4. Tests on simply supported sand asphalt beams (11) at 77 F showed that there was considerable interaction between creep and fatigue. The amount of creep was minimized and more realistic conditions were simulated when the beams were supported on an elastic foundation. Both controlled stress and controlled strain types of loading were used.

5. The results of the tests on the beams supported on an elastic foundation provided further verification of the crack propagation law and of the fact that the starter flaw was indeed a constant for the material. Furthermore, the prediction of the fatigue life was independent of the method of loading (controlled strain or controlled stress). That effect is fully accounted for by changes in the stress-intensity factor due to changes in the load.

6. The experiments on asphalt concrete beams (12) supported on an elastic foundation also showed excellent correlation with the crack growth law. The asphalt concrete mix was a typical Ohio Department of Highways 404 mix containing 6.5 percent asphalt content with both 60-70 and 85-100 penetration asphalt.

7. Sand asphalt slabs 44 in. in diameter were tested on an elastic foundation by an MTS machine (7). The amount of cracking and the crack pattern were obtained by X-ray photography. The cracks originated at the bottom and grew radially. Eventually circumferential cracks that originate at the top appeared. The completion of the circumferential crack marked the end of the experiment. The crack pattern is shown in Figure 4.

8. The results provided further verification of the validity of the crack propagation law: $dc/dN = AK^4$. The value of A was approximately the same as the value determined for the sand asphalt beams of the same composition as the slabs, showing that A was indeed a property of the material.

The foregoing principles can now be applied to the design and analysis of pavements.

OUTLINE OF METHOD OF ANALYSIS AND DESIGN OF MULTILAYERED PAVEMENT SYSTEMS AGAINST FATIGUE DISTRESS

Inevitably pavement design against fatigue distress involves many complex and interrelated factors. The foregoing theoretical and experimental work established that the rate of crack propagation in a pavement could be expressed in terms of the properties of the component layers and the geometrical and boundary conditions. Thus, any variation in the material properties during the life of the pavement is automatically accounted for, providing that such variation can be ascertained in a quantitative sense.

Changes in Young's modulus E and Poisson's ratio ν of the asphaltic layer due to temperature, speed, and aging would also influence the fatigue behavior of the pavement with regard to the stress distribution and the effect on the crack growth constant A. Those effects must be ascertained with proper consideration to the environmental and climatic conditions. Similarly, the effects of changes in the subgrade support due to moisture content variation must be estimated.

Figure 1. Stress-deformation at crack tip.

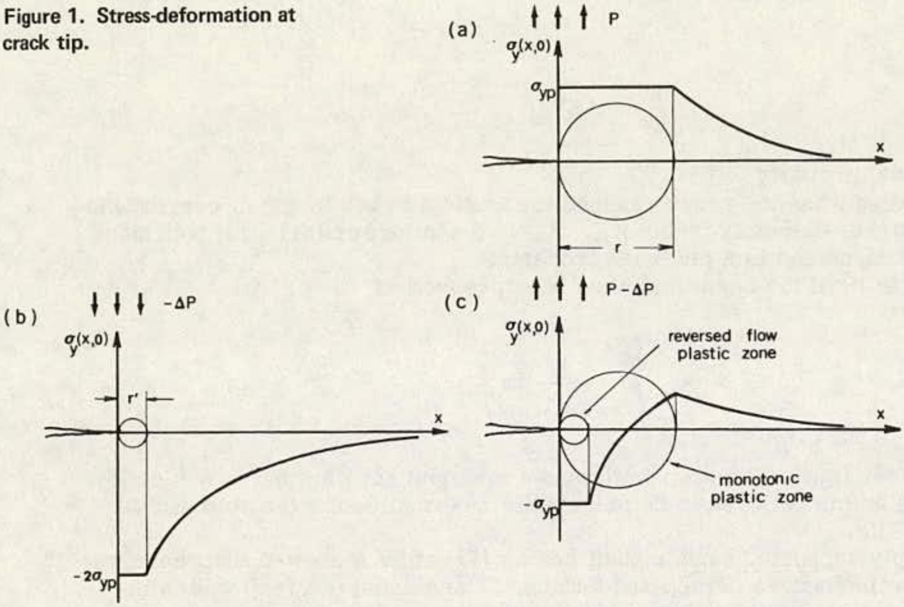


Figure 2. Modes of deformation of crack.

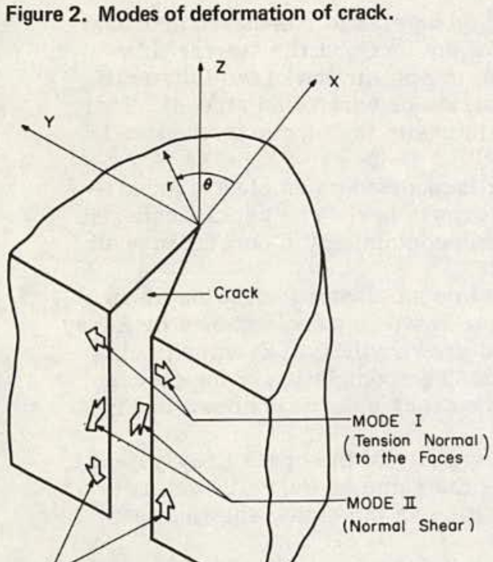


Figure 3. Dependence of square of stress-intensity coefficient on relative crack depth.

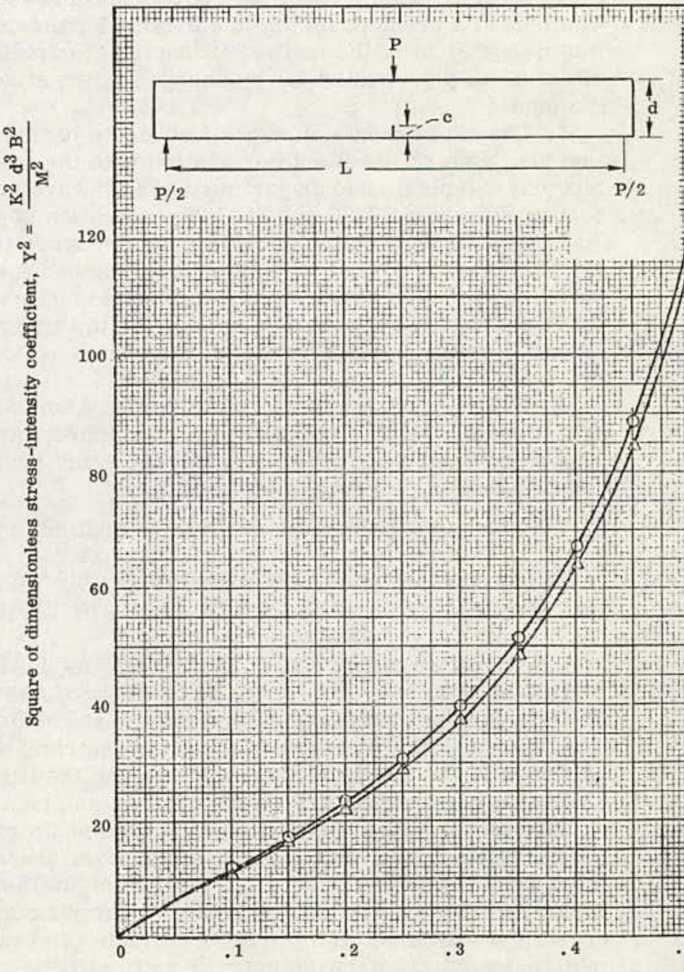
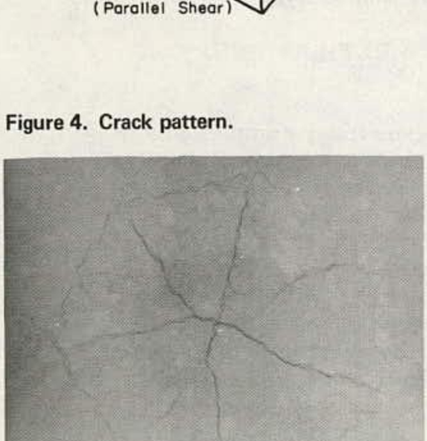


Figure 4. Crack pattern.



However, a number of important factors contributing to the fatigue life of a pavement have not been considered. Foremost among those are the cumulative effects of random loading and the effects of the interaction of rutting on cracking. Both of those topics are now the subjects of continuing research at Ohio State University. Recent results show that the effects of random loading can be largely accounted for by a simple modification of the concepts given above.

Of secondary importance is the effect of rest periods. Laboratory tests conducted at room temperature show that rest periods may be beneficial to fatigue due to healing. In practice, however, healing may not take place because of dust and water that may enter through the top cracks.

Within those concepts and within those limitations the following method of analysis and design is proposed.

Materials Characterization

1. Determine from laboratory tests which materials will be used for the base course and subgrade, and select a suitable asphaltic mix for the surface layer. Evaluate from standard test methods the properties E and ν for each layer and the modulus of subgrade reaction k as a function of temperature, frequency, aging index, and moisture content where applicable.
2. From fracture tests on beams of the same asphaltic mixture as the surface course, determine the critical stress-intensity factors K_{1c} and K_{2c} as functions of temperature, frequency, and aging index.
3. From fatigue tests on the asphaltic beams supported on an elastic solid, determine the constants A_1 and A_2 in the crack propagation law $dc/dN = A_1K_1^4 + A_2K_2^4$ as functions of temperature, frequency, and aging. Determine also the range and distribution of the experimental constant c_o.

Analysis and Evaluation

The basic principle of the fatigue analysis is that damage will be proportional to the average of the fourth power of the rises and falls in the load-time history of random loading (13) as shown in Figure 5. The exact damage per passage of axle load dc/dN is expressed by

$$\frac{dc}{dN} = A_1K_1^4 + A_2K_2^4$$

where K_1 and K_2 = average of the rises and falls of the stress-intensity factors corresponding to the peak-to-trough rises and falls in the K-time histories.

1. Obtain the components of the stress-intensity factors K_1 and K_2 for a pavement slab for any configuration of wheel loads, for any size and location of crack, and for any combination of the thicknesses and material properties of the component layers. Typical influence lines for K_1 and K_2 for a semi-infinite crack for a full-scale pavement are shown in Figure 6.
2. Determine the fatigue crack propagation in the pavement by determining the size and distribution of starter flaws c_o in the pavement. For a typical pavement the lateral distribution of wheel loads may be assumed to be as shown in Figure 7. Evidently the fastest crack growth will take place under the greatest concentration of loading because the stress-intensity factor is highest there. Thus, the cracking will be assumed to be primarily along the wheel track and perpendicular to it as shown in Figure 7.
3. Determine the size of starter cracks along the line of cracking by applying statistical analysis to laboratory tests on specimens prepared in the laboratory or cut out from the pavement at regular intervals or by cutting out continuous specimens in the wheel track over a distance equal to the shortest distance over which the distribution of c_o will be repetitive. Only the upper range of c_o values will be of practical interest because for exceedingly small values the crack will not propagate to the surface in the design period of the pavement.

Figure 5. Load-time and K factor-time history of random loading.

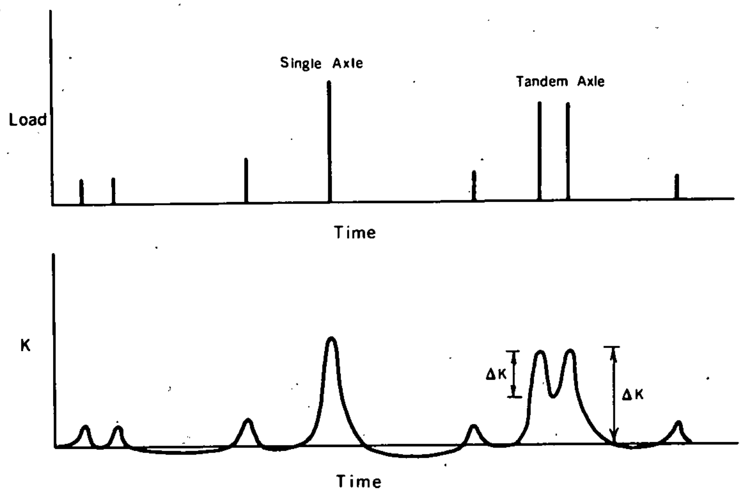
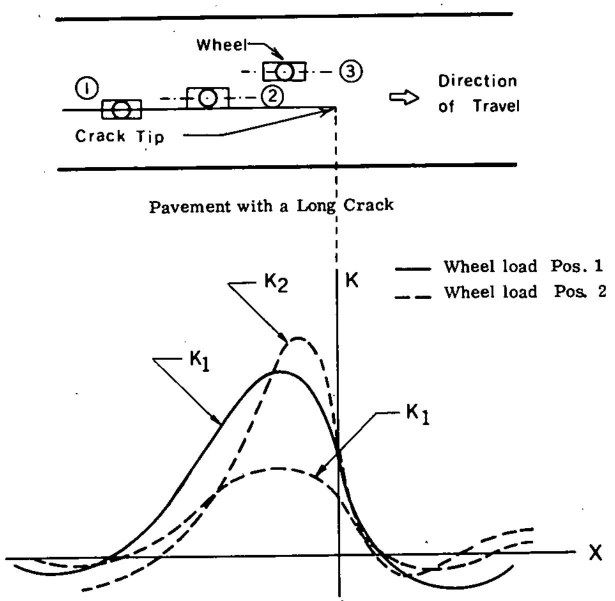


Figure 6. Typical influence lines for K_1 and K_2 for semi-infinite crack.



4. Confining attention to the growth of the cracks in a small representative section of the pavement, obtain the value of K_1 and K_2 for each crack. For simplicity it will be assumed that the distribution of the cracks is such that there is no interaction.

5. Integrate the rate of crack propagation numerically for 1 passage of the load train, and obtain the increment of damage. Damage is defined as the length of crack inclusive of the plastic zone formed at the crack tip or, for simplicity, as the length of crack. Thus, the increment of damage Δc is

$$\Delta c = \sum_{p=1}^{n} \left[A_1(\Delta K_{1_p})^4 + A_2(\Delta K_{2_p})^4\right]$$

where K_{1_p} and K_{2_p} = averages of the rises and falls of the influence lines for K_1 and K_2 respectively of the stress-intensity factor for each crack for the pth axle load.

6. Increase the length of the cracks, and compute K_1 and K_2 again. If the maximum values of K_1 and K_2 exceed K_{1c} and K_{2c}, the critical values of K, for any axle loads in the load train, the cracks will propagate rapidly and the pavement will be considered to have failed. If not, then increase each crack length by the increment of damage Δc, and repeat the procedure.

7. For each location of the assumed starter flaws, compute at reasonable intervals the length of longitudinal and transverse crack, and determine the total area of cracking as the product of the sum of the longitudinal cracks and the average length of the transverse cracks.

Thus at regular intervals the maximum values of K_1 and K_2 and the total area of cracking will be known. If K_1 and K_2 exceed the critical values of K_{1c} and K_{2c}, then the pavement will fail by rapid crack extension. On the other hand, K_1 or K_2 may never exceed K_{1c} or K_{2c} even if the length of cracks becomes exceedingly long. In such cases it is convenient to adopt the suggestion of Zube and Skog that, when the alligator type of cracking exceeds 10 percent of the total area, the pavement should be considered to have failed.

In this discussion, longitudinal and transverse cracks have been considered as the primary mode of crack propagation. In reality, of course, cracks in other directions and parallel to the main cracks will also develop rapidly into the well-known alligator pattern (hexagonal-shaped cracks). The alligator pattern may be deduced directly from the Griffith theory of fracture, which requires that the total surface energy expended to form new surfaces must be a minimum. It seems mathematically that this condition is best satisfied for hexagonal-shaped cracks rather than triangular-, quadrilateral-, or pentagonal-shaped cracks.

Once the primary mode of crack propagation has reached an advanced stage, the secondary and tertiary modes of cracking leading to the completion of the alligator pattern will follow in rapid succession. Thus, the definition of the failure criteria based on the primary mode of crack propagation is sufficiently accurate to obtain an estimate of the service life. Should the pavement design prove to be unsatisfactory, a new design must be made and the analysis for the service life repeated.

Load Equivalency Factors

From the theoretical relation $dc/dN = AK^4$ and the fact that K is proportional to the load P, the load equivalency factor for single axle loads is proportional to the fourth power of the load (but may be somewhat higher because of the effects of random loading). The load equivalency factor for tandem axles depends on the spacing of the axles and the shape of the influence line of K as the loads move across the crack.

As an illustration, consider 2 pavement sections, shown in Figures 8 and 9, that have about the same structural capacity (by the AASHO design method), but different relative stiffnesses. The influence lines for K for an 18-kip single axle and a 36-kip tandem axle load moving directly over a longitudinal crack 2 ft long are also shown in Figures 8 and 9.

Figure 7. Typical lateral distribution of wheel loads in pavement and assumed idealized random distribution of cracks.

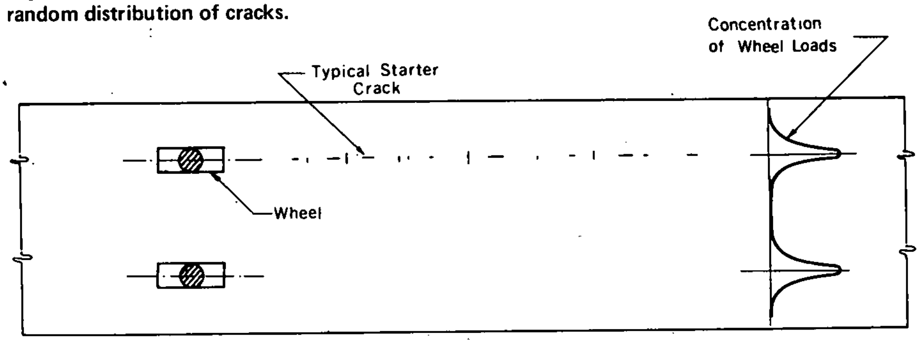


Figure 8. Load equivalency factor for high relative stiffness pavement.

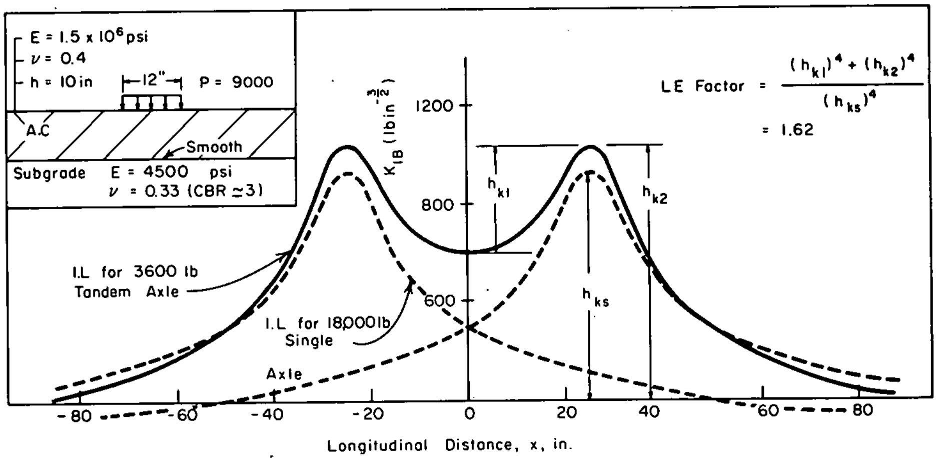
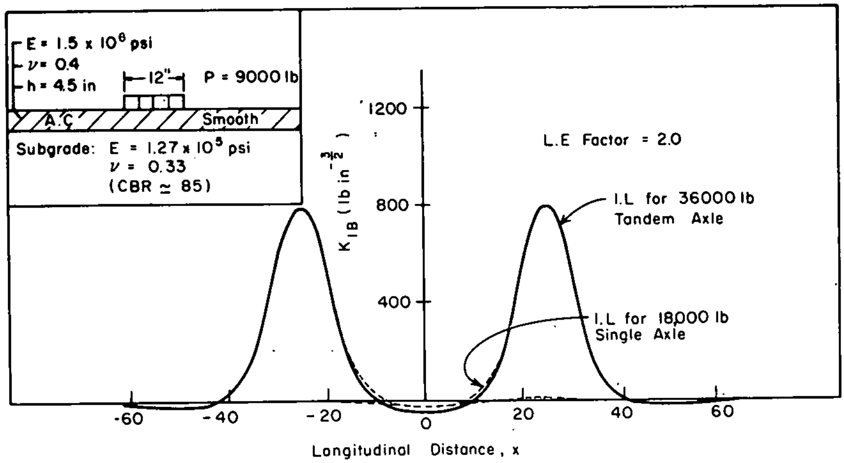


Figure 9. Load equivalency factor for low relative stiffness pavement.



The load equivalency factor, defined as the destructive ratio or crack ratio produced by 1 passage of the axle load as compared to an 18-kip single axle load can be obtained by taking the ratio of the fourth power of the rises and falls in the influence lines for K. Applying that criterion shows that the factor changes from 1.62 to 2.0 as the relative stiffness of the pavement decreases.

ADVANTAGES OF MECHANISTIC METHOD

1. The prediction of the fatigue life of pavements from laboratory tests is independent of the mode of loading or the specimen geometry. Those effects are completely accounted for by the stress-intensity factor.
2. Failure is defined realistically as follows: (a) failure by rapid crack propagation occurs when the stress-intensity factor for a particular crack approaches the critical stress-intensity factor and is most likely to occur under very heavy loads, thick asphalt layers, and brittle mixes; or (b) failure occurs when the area of cracking (obtained directly from the computations) exceeds 10 percent of the total area of the pavement.
3. Load equivalency factors for any type of loading can be obtained by a simple and reasonably accurate procedure.
4. Statistical variations are explained by the basic concepts of the method.
5. The effect of variables other than the bending stress, e.g., the modulus of the foundation and the thickness of the asphaltic layer, is accounted for.

DISADVANTAGES OF MECHANISTIC METHOD

1. To predict the area of cracking requires that the K-value be obtained for the actual crack pattern. However, solutions have been obtained only for 2-dimensional cases and certain simplified crack patterns such as a semi-infinite crack, a semicircular crack, or a very short crack for 3-dimensional cases. That represents the main difficulty in applying the method to obtain realistic crack patterns in pavements. Some type of numerical or finite element solution is required.
2. The computational time is at present lengthy, but simplifications could be introduced to reduce this to an acceptable time.

FUTURE DEVELOPMENT

1. A solution to obtain the K-value for all types of crack patterns is required.
2. There is need for the development of a theoretical solution to account for the effects of the interaction between cracking and rutting due to repeated loading.
3. The effects of random loading and of the various properties of the asphaltic mixture on fatigue need further investigation. Some of this work is being done at Ohio State University.

CONCLUSIONS

1. The crack propagation theory, $dc/dN = AK^4$, satisfactorily explains the fatigue performance of bituminous pavements; the stress-intensity factor K is the dominant parameter controlling crack growth, and A is a constant of the bituminous material. The theory applies only for plane strain conditions; that is when the thickness h of the bituminous slab exceeds $1.25\ (K_{1c}/\sigma_y)^2$, where K_{1c} is the critical value of K_1, and σ_y is the yield strength.
2. The critical stress-intensity factors K_{1c} or K_{2c} are the failure criteria for low temperatures or thick slabs. For higher temperatures or thin slabs, failure may be considered to have taken place when about 10 percent of the total surface area is cracked.
3. The load equivalency factor, defined as the destructive ratio when compared to an 18-kip single axle load, is proportional to the fourth power of the axle load for single axle loads; but, for tandem axle loads of equal magnitude, it is dependent on the spacing between the axles and the relative stiffness of the pavement. The load equivalency factor can be determined by taking the average of the fourth power of the peak-to-trough rises and falls in the K-time history.

4. The bending stress σ^* in the bituminous slab is not a sufficiently good parameter to describe the fatigue behavior of pavements, for it cannot account for the cracking and the subsequent redistribution of the stress. The stress-intensity factor K fully accounts for those effects; in general, K is a function of the bending stress, the crack length in the pavement, the relative stiffness of the pavement, and the geometrical and boundary conditions.

5. Finally, when one knows the traffic loading and the variation of material properties E, ν, A, K_{1c}, and K_{2c} due to temperature, moisture, and aging index during the service life of the pavement and also the magnitude and distribution of the starter flaws c_o, a reasonable estimate of the service life can be obtained by the method outlined.

ACKNOWLEDGMENT

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REFERENCES

1. McEvily, A. J., Boettner, R. C., and Johnston, T. L. On Foundation and Growth of Fracture Cracks in Polymers. In Fatigue, An Interdisciplinary Approach, Syracuse Univ. Press, 1964.
2. Paris, Gomez, and Anderson. A Rational Analytical Theory of Fatigue. In The Trend in Engineering, Univ. of Washington, 1961.
3. Paris, P., and Erdogan, F. J. A Critical Analysis of Crack Propagation Laws. Jour. Basic Eng., Trans., ASME, Ser. D, Vol. 85, 1963.
4. Paris, P. The Fracture Mechanics Approach to Fatigue. In Fatigue, An Interdisciplinary Approach, Syracuse Univ. Press, 1964.
5. Rice, J. Proc., Int. Conf. on Fracture, Sendai, Japan, 1965.
6. Gross, B., and Srawley, J. E. Stress-Intensity-Factors for Three-Point Bend Specimens by Boundary Collocation. NASA, TND-3092.
7. Ramsamooj, D. V. The Design and Analysis of the Flexibility of Pavements. Ohio State Univ., PhD dissertation, 1970.
8. Williams, Ang, and Folias. The Bending Stress in a Cracked Plate on an Elastic Foundation. Jour. Appl. Mech., Dec. 1967.
9. Srawley and Roberts. Plain Strain Fracture Toughness. In Fracture, An Advanced Treatise (Liebowitz, H., ed.), Vol. 4, 1969, Ch. 2.
10. Majidzadeh, K., Ramsamooj, D. V., and Fletcher, T. A. Analysis of Fatigue of Sand Asphalt Mixtures. Proc., AAPT, 1969.
11. Majidzadeh, K., Kauffmann, E. M., and Ramsamooj, D. V. Application of Fracture Mechanics in the Analysis of Pavement Fatigue. Proc., AAPT, Vol. 40, 1971.
12. Majidzadeh, K., Kauffmann, E. M., and Saraf, C. L. Analysis of Fatigue of Paving Mixtures From the Fracture Mechanics Viewpoint. Ann. Mtg. ASTM, June 1971.
13. Ramsamooj, D. V., Majidzadeh, K., and Kauffmann, E. M. The Design and Analysis of the Flexibility of Pavements. 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, Sept. 1972.

Discussion

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The authors have attempted to apply the theory of classical fracture mechanics for predicting the fatigue life of flexible pavements. Most of the material in the paper is concerned with the review of the elementary concept of fracture mechanics based on the stress-intensity factor parameter. The authors conclude that the stress-intensity factor is the parameter controlling crack growth in the pavement. However, the experimental data (13) quoted by the authors appear inconclusive. Therefore, the writer wishes to raise several questions with regard to the contents of the paper.

In the abstract of the earlier paper (13), the authors define the fatigue failure criterion of pavements as the time for the stress-intensity factor of the longest crack to reach its critical value at which rapid crack propagation occurs. That is in contrast to the crack growth data shown in Figure 7 of that paper (13) in which the crack tends to reach a subcritical crack length rather than rapid unstable crack propagation.

The authors use the word "damage" very loosely. It appears that damage and crack growth are used synonymously. In collecting crack growth data, it is essential to report the size of the crack opening, which could range from, say, 10^{-1} in. (nonmetals) to 10^{-6} in. (high-strength alloys). Moreover, the classical fracture mechanics theory is not a theory of cumulative damage.

Recent (14, 15) and previous (16) works on the direction of crack initiation have shown that there is only one fundamental mode of crack propagation, namely, that the crack runs in a plane perpendicular to the direction of maximum stress (16). The classical concept of mode 2 crack extension is inadequate because the crack does not run directly ahead. A more refined theory based on the stationary value of the strain energy density factor S (14, 15) indicates that the crack runs in the direction at which S reaches the critical value S_c. In pavement studies one has to treat the mixed mode crack problem involving a combination of at least k_1 and k_2 because the cracks do not run in a straight line.

The linear fracture mechanics theory is not restricted to the definition of small-scale yielding as stated by the authors where the zone of plasticity is small in comparison to the crack length. It has been shown that the elastic stress-intensity factor and the so-called "plastic stress-intensity factor" (18) do not differ significantly even though the plastic zone size may be as large as half the crack length.

The basic assumption of the crack growth relation used by the authors is an empirical power-law relation obtained from crack growth data on metal alloys (17). The fourth-power exponent is to be questioned because we know that the exponent n can vary from 2 to 100 depending on the material, the environmental conditions, and the range of cyclic loading. For example, in the range of 10^{-6} to 10^{-4} in./cycle, n normally varies from 2 to 10. Moreover, the exponent can deviate greatly from 4 if the cyclic range is varied below and above 10^{-6} to 10^{-4} in./cycle. Those observations have been made on metallic materials such as aluminum, steel, and titanium and on nonmetallic materials such as plexiglass and other thermoplastic materials. It is an open question as to whether the fourth-power exponent or any other value should be used for predicting the fatigue life of pavement structures. Experimental data shown in the references listed by the authors were based on simply supported beams of sand asphalt material tested at 23 F. It is not clear whether all the crack growth data in Figures 8 to 10 (13) were or were not taken from the beam-bending specimen where the crack is only partially through the thickness. It should be recalled that the fourth-power relation was originally established from data on metal specimens with a through crack running at both ends. Moreover, the analytical results of a cracked plate on an elastic foundation also correspond to a through crack and not an edge crack. Therefore, it is the opinion of the writer that the experimental data presented by the authors are insufficient to establish prediction relations for the fatigue life of pavements.

The authors state that "linear fracture mechanics is always applicable," based on the 1.25 $(k_1/\sigma_y)^2$ relation. One must remember that the concept of size limitation such as 2.5 $(k_1/\sigma_y)^2$ was established on the basis of a through crack in a homogeneous material with a reasonably well-defined yield stress. In the case of pavement material, the crack may not penetrate through the entire pavement, and, furthermore, one must carefully define what is meant by σ_y for a multiphased material that is also time dependent.

Based on the evidence presented by the authors, the discussion given above suggests that there is now inadequate justification for the application of fracture mechanics to the prediction of fatigue life of pavements based on a simple interpretation of the fracture mechanics theory using equations borrowed from studies on metal alloys. It should be made clear, however, that the concept of fracture mechanics can be used to analyze pavement materials as in the case of composite systems. Successful application of the theory depends on the ability of the analyst to come up with a realistic analytical model catering to the specific problem at hand. What has been developed for metals may not necessarily apply to pavement materials. There is no doubt that additional work must be done on defining the service loading and environmental conditions under which fatigue crack growth takes place in pavements.

REFERENCES

14. Sih, G. C. A Special Theory of Crack Propagation. In Methods of Analysis and Solutions to Crack Problems (Sih, G. C., ed.), Wolters-Noordoff Publishing Co., 1972.
15. Sih, G. C. Some Basic Problems in Fracture Mechanics and New Concepts. Symp. on Fracture and Fatigue, George Washington Univ., May 1972.
16. Erdogan, F., and Sih, G. C. On the Crack Extension in Plates Under Plane Loading and Transverse Shear. Jour. Basic Eng., Vol. 85, p. 519, 1963.
17. Paris, P. C. Growth of Cracks Due to Variations in Load. Lehigh Univ., PhD dissertation, 1962.
18. Hilton, P. D., and Hutchinson, J. W. Plastic Intensity Factors for Cracked Plates. Jour. Eng. Fracture Mech., Vol. 3, 1971, pp. 435-451.

Closure

The authors wish to thank Fang for his review and discussion of our paper. We fully concur that fatigue analysis of pavement systems using fracture mechanics requires consideration to complex service loading and environmental conditions and is much more involved than results presented in our paper. The data we presented are for a simplified laboratory pavement model with well-defined material characteristics and geometrical conditions. We regret to note that Fang in discussing the limitation of our results apparently assumes that we used a composite or multilayered asphaltic pavement system. We have clearly indicated that the results are obtained from a pavement model resting on elastic foundation.

Fang's comment on our use of the concept of damage is irrelevant. We have never stated nor suggested that fracture mechanics is a cumulative damage law. Furthermore, we fully disagree with his statement, "There is inadequate justification for application of fracture mechanics to the prediction of fatigue life of pavements . . . using equations borrowed from studies on metal alloys."

Stress dependency of the damage rate and its sensitivity to specimen geometry is a well-known and accepted concept. The Paris equation used in our study is only another form of representing the rate of damage due to fatigue loading.

We concur with Fang that this equation is a semi-empirical formula and n might be other than 4. It is also granted that test data from metal alloys and certain paving mixtures show a variation from 2 to 5 (4, 5, 6, 7). The important fact is that, for the sand asphalt material used in the Ohio State University research projects, it was found experimentally that the fourth-power relation best fits laboratory data. Figures 8 and 9 (13) clearly show that the data were obtained from tests on slabs resting on an elastic foundation, while it is also shown in Figure 10 that the data come from beams and slabs, shown there as a comparison.

There is absolutely no contradiction between the stated criterion for fatigue failure, defined as the time for the stress-intensity factor at the tip of the largest crack to reach its critical value or the time for the total area of cracking to exceed 10 percent of the area of the pavement surface, and the crack growth curve from a laboratory test on a slab with stationary load shown in Figure 7 (13). Obviously the criterion postulated refers to a real pavement system with moving loads.

It has been proposed by the authors (2, 3) that the exact damage per passage of axle load in a pavement, dc/dN, is expressed by

$$\frac{dc}{dN} = A_1K_1^4 + A_2K_2^4$$

where K_1 and K_2 are the average of rises and falls of the stress-intensity factors corresponding to mode 1 and mode 2 of crack extension. However, as Fang indicated, the use of energy density factor S seems more appropriate.

The idea of using the plastic stress-intensity factor is a sound one. However, at the time this study was being conducted, experimental evidence was not available to compare differences between plastic and elastic stress-intensity factors.

If indeed the concept of K can be shown to be applicable to larger plastic zones, it will greatly reinforce the application of fracture mechanics to analysis of pavement systems.

We have shown evidence from observations during the test and X-ray photographs (2, 3) that the cracks do penetrate through the entire pavement. That is why the theory for part-through cracks was used. The yield stress σ_y was measured experimentally versus rate of loading, and the results were reported (2).

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.

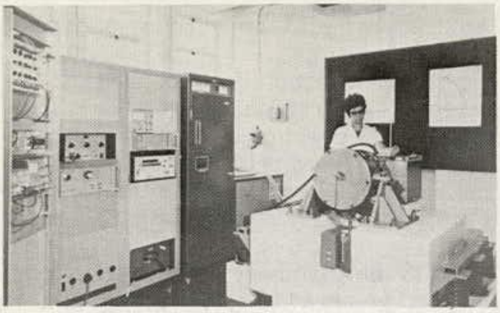


Figure 2. Loading of trapezoidal-shaped specimen.

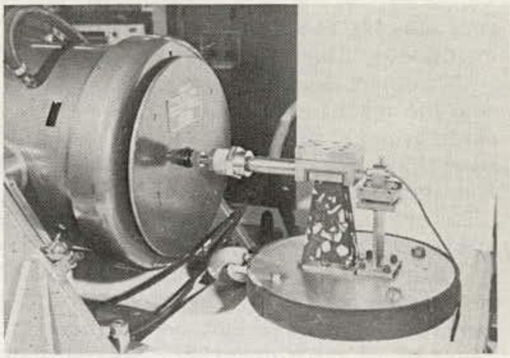


Figure 3. Laboratory and field peak stiffness values for asphaltic-concrete and gap-graded mixtures.

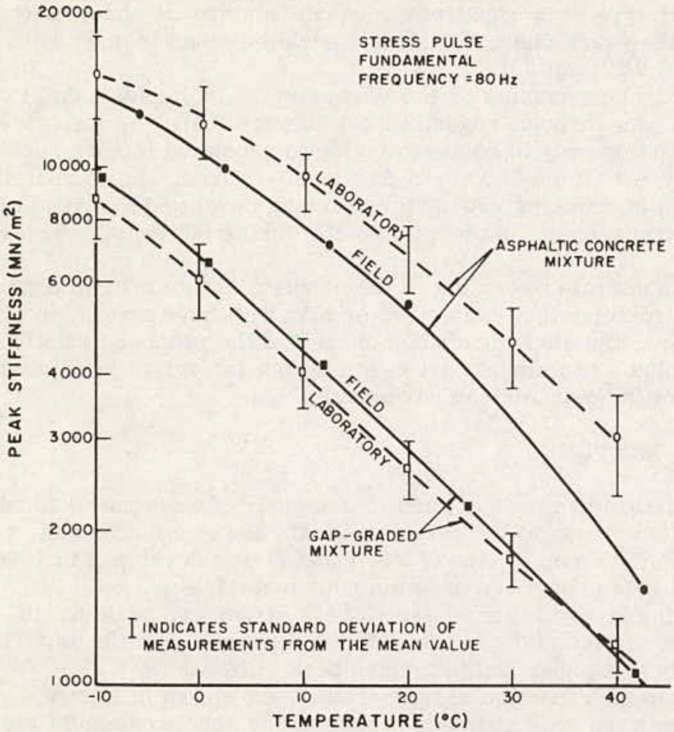
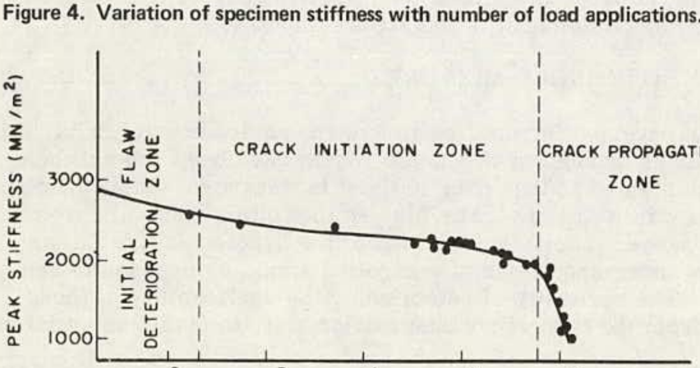


Figure 4. Variation of specimen stiffness with number of load applications.



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 $(N_s)^x_\epsilon$ 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, $(N_s)^{10}_\epsilon$ and $(N_s)^{50}_\epsilon$, 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, $(N_s)^x_\epsilon$, $(N_s)^{10}_\epsilon$, and $(N_s)^{50}_\epsilon$ 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

$$S = R \log N + S_1 \quad (2)$$

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.

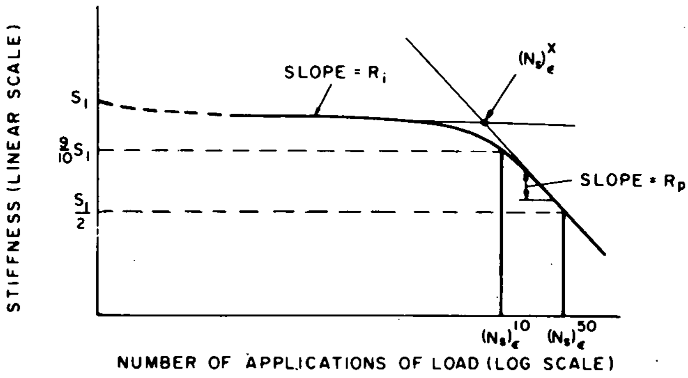


Figure 6. Variation of R_i and R_p with temperature.

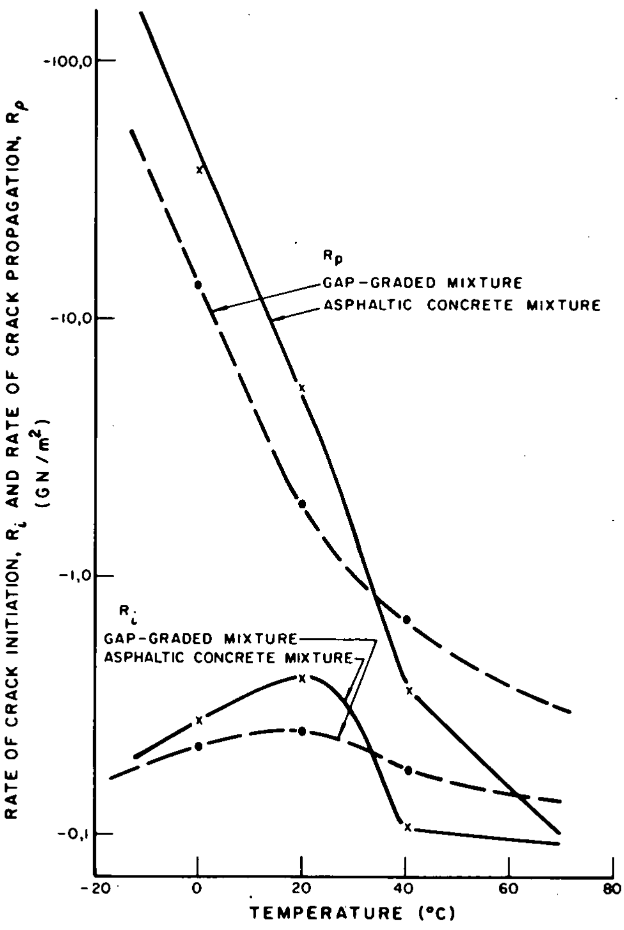


Figure 7. Service life strain for experimental pavement surface.

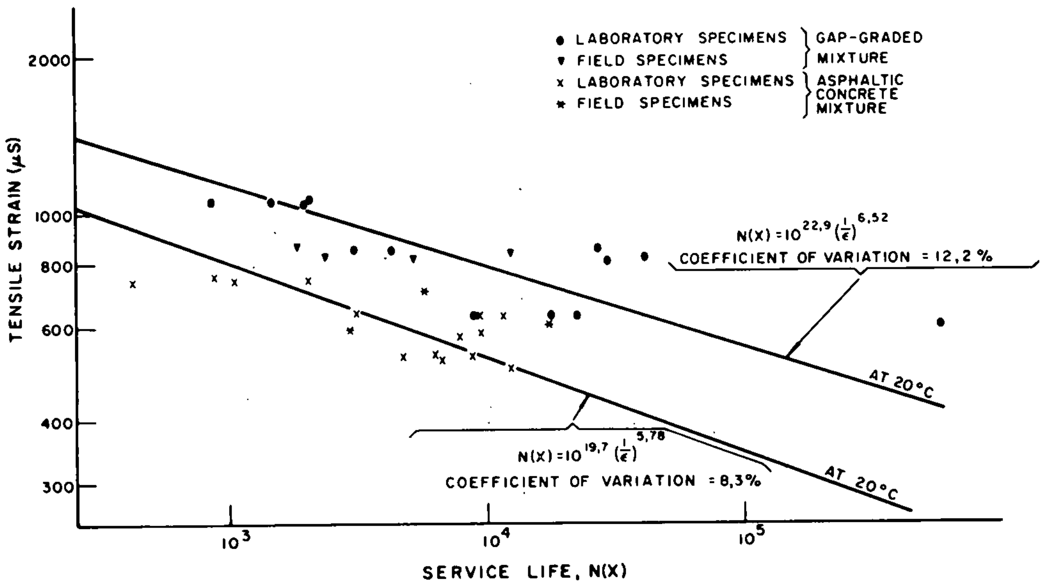
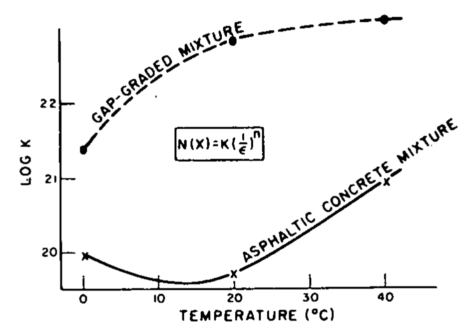
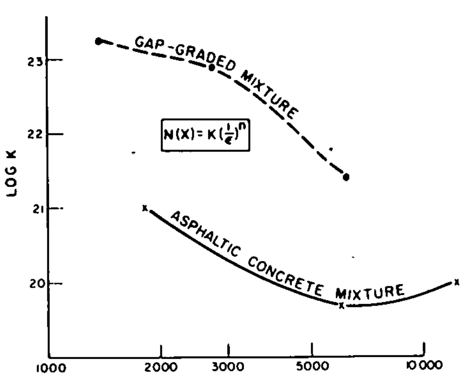


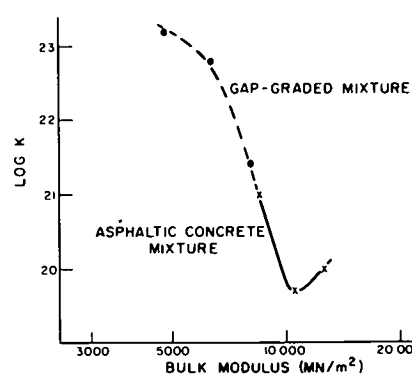
Figure 8. Variation of log K with temperature, peak stiffness, and bulk modulus.



(a) VARIATION OF LOG K WITH TEMPERATURE



(b) VARIATION OF LOG K WITH PEAK STIFFNESS



(c) VARIATION OF LOG K WITH 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.

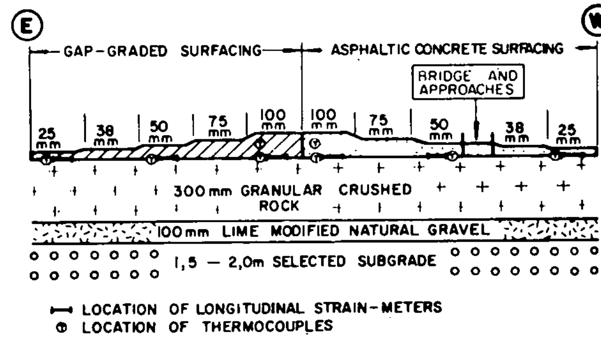


Figure 10. Load distribution of average daily count on experimental pavement.

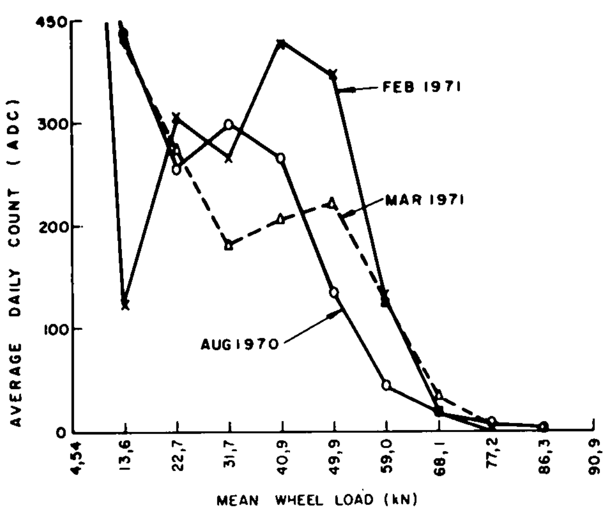
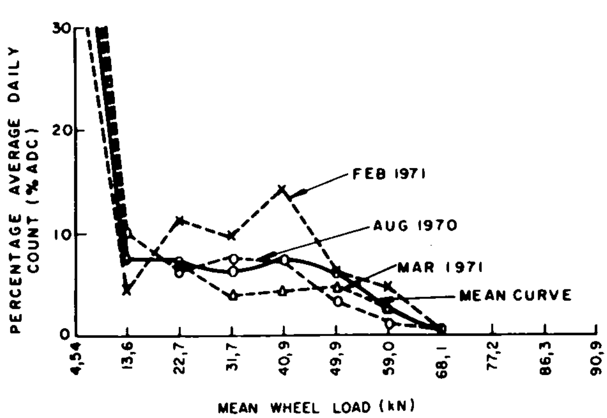


Figure 11. Load distribution of percentage of average daily count on experimental pavement.



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.

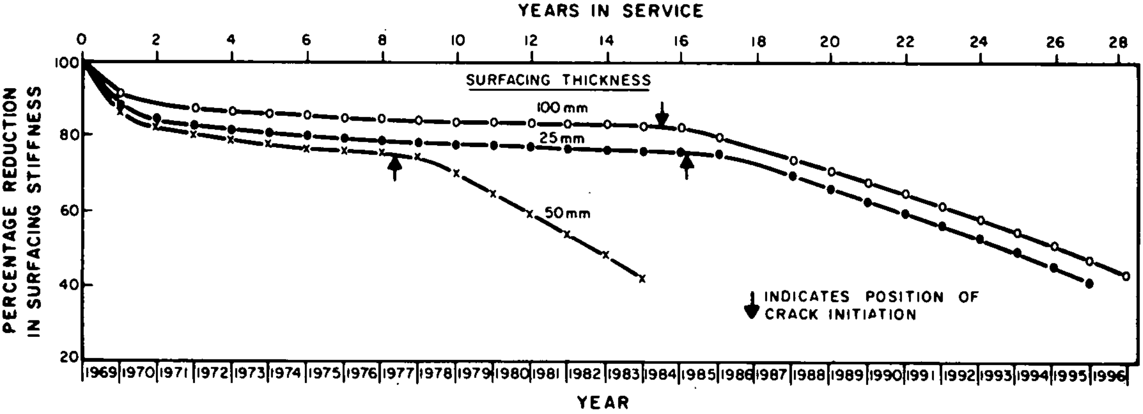
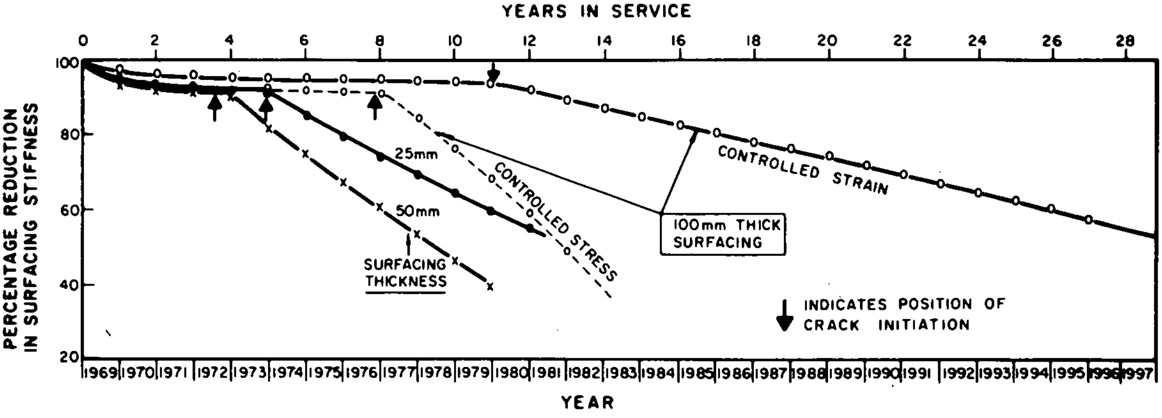


Figure 13. Reduction in asphaltic-concrete surface stiffness with number of years.



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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REFERENCES

1. Freeme, C. R. The Behaviour of Bituminous Surfacings in Asphalt Pavements. Univ. of Natal, PhD dissertation, Sept. 1971.
2. Rigden, P. J. The Challenge of the Seventies. Proc., Conf. on Asphalt Pavements for Southern Africa, Durban, South Africa, Aug. 1969.
3. Dehlen, G. L. Limitations of the Use of Thin Surfacings and Unbound Bases for Pavements. Proc., Conf. on Asphalt Pavements for Southern Africa, Durban, South Africa, Aug. 1969.
4. Non-Traffic Load Associated Cracking of Asphalt Pavements. Proc., AAPT, Vol. 35, Feb. 1966, pp. 239-357.
5. Hveem, F. N., and Carmany, R. M. The Factors Underlying the Rational Design of Pavements. HRB Proc., Vol. 28, 1948, pp. 101-136.

6. Porter, O. J. Development of CBR Flexible Pavement Design Method for Airfields. Symp. Trans., ASCE, Vol. 115, 1950, pp. 461-467.
7. Dehlen, G. L. An Investigation of Flexure Cracking on a Major Highway. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1962, pp. 812-820.
8. Dehlen, G. L. Results of Inspections and Level, Deflection and Curvature Measurements, for the First Few Years After Construction of the Foundation and Surfacings Experiments at Key Ridge and St. Helier in Natal. Natl. Inst. for Road Res., CSIR, South Africa, RS/4/67, 1967.
9. Williams, A. A. B., and Dehlen, G. L. The Performance of Full-Scale Base and Surfacings Experiments on National Route 3-1, at Key Ridge After the First Six Years. Proc., Conf. on Asphalt Pavements for Southern Africa, Durban, South Africa, 1969.
10. Monismith, C. L. Design Consideration for Asphalt Pavements. Proc., Conf. on Asphalt Pavements for Southern Africa, Durban, South Africa, Aug. 1969.
11. Deacon, J. A. Fatigue of Asphalt Concrete. Inst. of Transp. and Traffic Eng., Univ. of California, Berkeley, Grad. Rept., 1965.
12. Pell, P. S. Fatigue of Asphalt Pavement Mixes. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1967, pp. 577-593.
13. Kasianchuk, D. A. Fatigue Considerations on the Design of Asphalt Concrete Pavements. Univ. of California, Berkeley, PhD dissertation, 1968.
14. Epps, J. A. Influence of Mixture Variables on the Flexural Fatigue and Tensile Properties of Asphalt Concrete. Univ. of California, Berkeley, PhD dissertation, 1968.
15. Pell, P. S. Fatigue Characteristics of Bitumen and Bituminous Mixes. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1962, pp. 310-323.
16. Monismith, C. L. Asphalt Mixture Behavior in Repeated Flexure. Soil Mech. and Bitum. Mater. Res. Lab., Univ. of California, Berkeley, Rept. TE-66-6, 1967.
17. van der Westhuizen, M. M., and Freeme, C. R. Determination of the Elastic Moduli of Perspex Specimens Using Both Static and Dynamic Methods. Natl. Inst. for Road Res., CSIR, South Africa, RB/25/68, 1968.
18. Freeme, C. R. Flexural and Fatigue Behaviour of Bituminous Mixes Under Dynamic Loads. Natl. Inst. for Road Res., CSIR, South Africa, RB/1/69, 1969.
19. Freeme, C. R., and Marais, C. P. The Structural Behaviour of Bituminous Surfacings in an Experimental Asphalt Pavement. Proc., 3rd Int. Conf. on Struct. Des. of Asphalt Pavements, London, 1972, pp. 812-822.
20. Szendrei, M. E., and Freeme, C. R. The Computation of Road Deflections Under Impulsive Loads From the Results of Vibration Measurements. Proc., 2nd Int. Conf. on Struct. Des. of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Aug. 1967, pp. 141-149.
21. Szendrei, M. E., and Freeme, C. R. Road Responses to Vibration Tests. Proc., ASCE, Vol. 96, No. SM6, Nov. 1970, pp. 2099-2124.
22. Millard, R. L., and Marais, C. P. The Design and Installation of Strain Meters in the Base and Flexibility Sections of the S12/2 Road Experiment. Natl. Inst. for Road Res., CSIR, South Africa, RB/13/69, May 1970.
23. Williamson, R. H. The Calculation and Simulation of Temperature Variations in Pavements. Natl. Inst. for Road Res., CSIR, South Africa, RP/1/71.
24. Barber, E. S. Calculation of Maximum Pavement Temperatures From Weather Reports. HRB Bull. 168, 1957, pp. 1-8.
25. Wilson, E. L. Analysis of Prismatic Solids. Dept. of Civ. Eng., Univ. of California, Berkeley, 1968.
26. Schenck, H. M., Jr. Fortran Methods in Heat Flow. Ronald Press, New York, 1963.
27. Southgate, H. F., and Deen, R. C. Temperature Distribution Within Asphalt Pavements and Its Relationship to Pavement Deflection. Highway Research Record 291, 1969, pp. 116-131.

28. Finn, F. N. Factors Involved in the Design of Asphaltic Pavement Surfaces. NCHRP Rept. 39, 1967.
29. Schultz, B. R. Climate of South Africa: Part 8—General Survey. Weather Bureau, South Africa Dept. of Transport, 1965.
30. Freeme, C. R. The Development and Evaluation of a Portable Axle Weight Analyser. Natl. Inst. for Road Res., CSIR, South Africa, RB/2/72, 1972.
31. Charts for Flexible Pavements. Shell International Petroleum Co., London, 1963.
32. Brown, S. F., and Pell, P. S. Developments in the Structural Design of Flexible Pavements. Roads and Road Construction, May 1970, pp. 141-156.
33. The AASHO Road Test: Report 5—Pavement Research. HRB Spec. Rept. 61E, 1962.
34. Liddle, N. J. Application of AASHO Road Test Results to the Design of Flexible Pavement Structures. Proc., Int. Conf. on Struct. Des. of Asphalt Pavements, Ann Arbor, Mich., 1962, pp. 42-51.
35. Grant, M. C. Some Aspects of Load Strain Relationships as They Affect the Failure of a Pavement Structure. Natl. Inst. for Road Res., CSIR, South Africa, RC/7/70, June 1970.
36. Marais, C. P., and Burton, R. W. An Investigation to Evaluate the Bulk Specific Gravities of Premix Surfacing Material Compacted by the ASTM, Marshall Method and Cores Obtained From Umbilo Road Premix Surfacing Experiment. Natl. Inst. for Road Res., CSIR, South Africa, RB/8/70, June 1970.

Discussion

Yang H. Huang, University of Kentucky

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 $E_1 e/q$, 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.

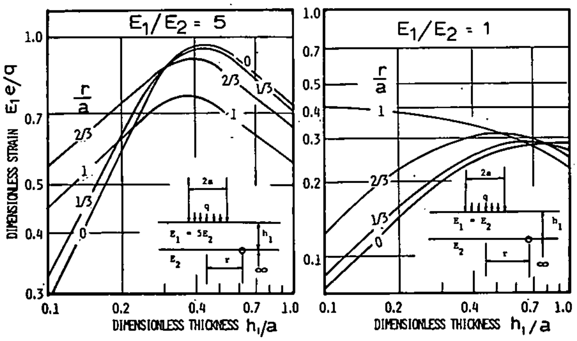

Figure 15. Effect of thickness and modulus ratio on critical tensile strain.

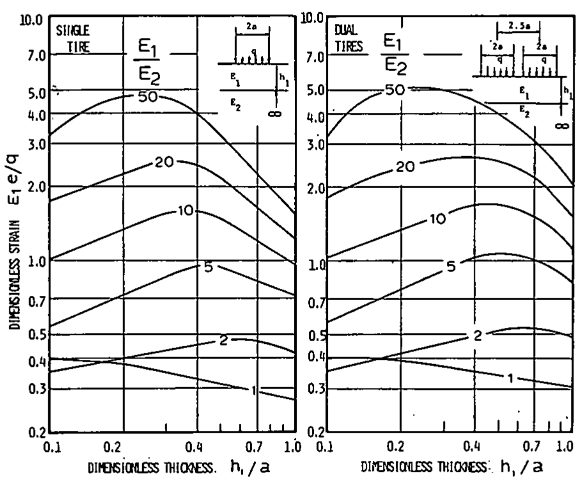

Table 4. Wheel configurations and tire pressures for various wheel-load categories.

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.

The following paper was prepared by Mr. Kingham for presentation at the Third International Conference on the Structural Design of Asphalt Pavements while he was a member of the Asphalt Institute staff. Since then, he has joined the Highway Research Board staff. Because Mr. Witczak, in an earlier paper in this Special Report, uses the fatigue criteria developed by Mr. Kingham, this paper in included for information and is reproduced from the conference proceedings with permission of the Executive Committee of the Third International Conference on the Structural Design of Asphalt Pavements.

FAILURE CRITERIA DEVELOPED FROM AASHO ROAD TEST DATA

R. Ian Kingham*

ABSTRACT

Theoretical models of pavement deformation behavior such as elastic-layered theory can only be used for design purposes when failure criteria are specified. Although such models can be used to predict stress and strain states, they in no way indicate whether the material in the pavement can withstand the predicted deformations. For elastic-layered theory, limiting values of strain or stress need to be defined before the theory can be used to assist practicing engineers in the design of asphalt pavements.

There is general agreement in the literature that horizontal tensile stress or strain at the bottom of a thick asphalt layer is the controlling criterion for design to prevent repetitive load cracking. Although such strains were not measured at the bottom of the asphalt layer at the AASHO Road Test, they can be inferred from a knowledge of the material characteristics and the measured deflections. Repetitive load cracking was observed to be the predominant mechanism of initial failure at the Road Test. Since the bituminous base sections provided a complete range of performance, from failures to survivors of over 1 million load repetitions it was possible to describe the strain history of these test sections in terms of performance.

The bituminous base sections fell into three performance classifications, depending upon whether they failed the first spring of testing, survived the testing period with a low serviceability rating or survived the testing without any change in serviceability. The horizontal tensile strain, horizontal tensile stress and vertical strain on top of the subgrade data were computed for each test section in each performance classification. Asphalt moduli for a wide spectrum of deflection measurements were input into the stress and strain computations. Moduli values were determined from dynamic loading in compression. Subgrade moduli were inferred from the deflection measurements.

The results of the elastic-layered computations showed that there were indeed large differences in horizontal tensile strain, horizontal tensile stress and vertical strain in the subgrade, depending upon the performance classification. Secondly, the level of strain or stress for each performance classification was a function of the asphalt base stiffness at the asphalt layer bottom. From the horizontal strain results it was apparent that asphalt pavements can tolerate higher strains at lower stiffnesses.

The horizontal tensile strain and stress relationships with asphalt stiffness were converted into "load repetition to failure" relationships by relating two performance classifications to the number of load repetitions to failure. A log-log relationship was assumed. The resulting family of "fatigue-like" curves for a range of asphalt stiffnesses has been used by Witczak and is the subject of another paper to this conference.

INTRODUCTION

The development of failure criteria described in this paper was undertaken to complete one structural model required in a pavement management system.[1] A pavement management system has four major subsystems which in turn may be further subdivided. These major subsystems are design, construction, maintenance and the processing of information. The information subsystem is concerned with all aspects of pavement management and provides the source of feedback into the other subsystems.

The design subsystem is concerned with the material selection and thickness design of pavements and their interrelationship with the many other factors affecting road performance. To describe the pavement design subsystem, it may be further divided into subsystems that relate to mechanisms of failure. Haas, Kasianchuk and Terrel, in a paper to this conference,[2] have outlined the research needs required to develop the fatigue, rutting and thermal fracture subsystems. Fatigue and rutting are acknowledged to be load associated, whereas thermal fracture is more closely associated with cold temperatures and secondly to load. This paper is concerned with developing failure criteria for use in the fatigue subsystem. Major emphasis is given to Full-Depth asphalt pavements and thick asphalt bases.

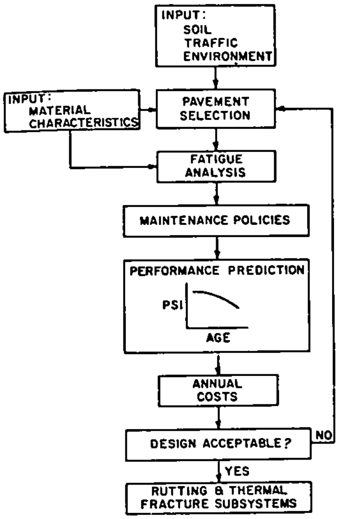

Fig. 1 FATIGUE SUBSYSTEM.

BACKGROUND

A major part of the fatigue subsystem shown in Figure 1 is the fatigue analysis which requires the development of a structural model. Miner's hypothesis provides such a model which can be used to predict failure or to design pavements for a given life. This hypothesis requires the computation of stress or strain from a behavior model and a knowledge of the load applications to failure for each level of stress or strain. The stress and strain computations for a given pavement can now be easily computed using one of the several available computer solutions to the elastic layered system. The Chevron Asphalt program[3] was used by this author. Finn and Hicks[4] have shown that the measured deflection and strains are reasonably close to those computed by the elastic layered system. The author has made other studies himself with data obtained from test roads in Brampton, Ontario and in Colorado to verify that deflections, vertical strains and radial strains can be reasonably computed from elastic layered theory.

For the determination of load repetitions to failure as a function of pavement deformation, Dormon and Metcalf[5] have rationalized that the horizontal stresses and strains at the bottom of the asphalt layer and the vertical strain in the subgrade are critically related to pavement performance. Deacon and Monismith[6] and Pell[7] have used a flexural test of asphalt mixtures in the laboratory to determine relationships between radial strain and load repetitions to failure. They have shown to this author that the scatter obtained from the laboratory test can be considerable and that a large number of tests are required to determine the relationship for a specific asphalt mixture. In another paper to this conference,[8] this author describes the problem of establishing a family of laboratory fatigue curves that consider the asphalt mixture stiffness. Another approach to determining the repetitions to failure as a function of stress or strain is to infer values from full-scale field tests. Although the AASHO Road Test[9] did not produce measured values of strain as a function of load repetitions to failure, it does provide a substantial number of thick asphalt base sections from which stress or strain criteria can be inferred.

Limitations of using AASHO Road Test data for the purpose of deriving failure stresses and strains were recognized at the outset. There was no check on the computed strains and stresses using measurement data. Secondly, failure at the AASHO Road Test was described by an unsatisfactory level of present serviceability index. Present serviceability index was not developed to represent the effects of any one particular mode of failure. However, from the author's observations while employed at the AASHO Road Test, failure initiated with cracking in the wheel path. Rutting and shear failure were only evidenced in advanced stages of failure after the cracking had occurred. It is believed that stress and strain failure criteria as developed from the AASHO Road Test data represent very closely failure stresses and strains for a fatigue mechanism of failure. Such curves, however, are not true fatigue curves and hence in this paper will be referred to as load repetition curves.

HYPOTHESIS

To obtain an understanding of performance and deformation behavior of the bituminous base sections, a detailed study was made of performance and deflection measurements. For each test section, present serviceability index and deflection were plotted for each measurement date to show annual trends. Figure 2 presents a typical example. It was readily apparent that the peak deflections for the bituminous base sections were to be found in the summer during periods when temperatures were at their maximum. This trend was in contrast to the granular base, where deflection measurements peaked in the spring. However, for both the granular base and bituminous base designs, failures took place predominantly in the spring. Almost no cracking occurred during the summer peak deflection periods. Some rutting, however, was measured during the summer and was considered tolerable by the Road Test staff in thicknesses adequate to resist cracking. It was hypothesized that critical strains must be a function of the asphalt layer stiffness since cracking did not occur during the summer.

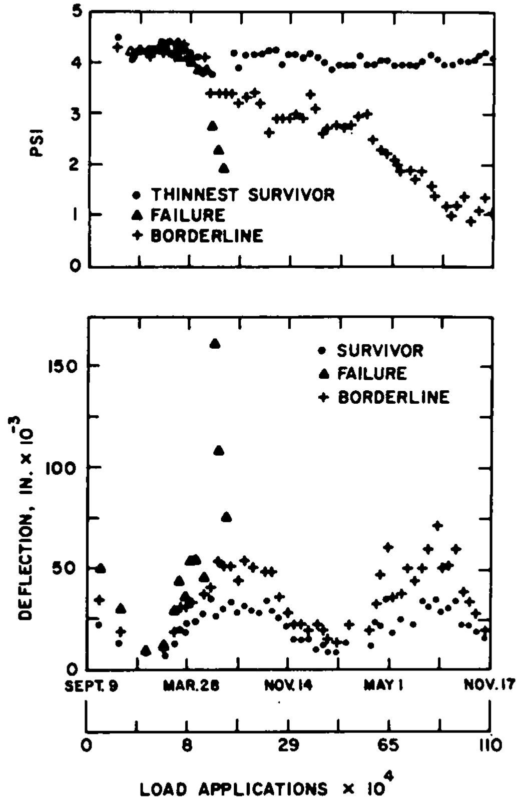

Fig. 2 TYPICAL PSI AND DEFLECTION TRENDS BY PERFORMANCE CLASSIFICATION.

The AASHO Road Test included bituminous base sections under three weights of loading -- 12,000, 22,400 and 30,000 lb. axle loads. For each load there appeared two or three trends with time as shown in Figure 2. For all loads one test section failed during the first spring of testing, having carried approximately 121,000 load repetitions throughout the previous fall and winter. These are described as "failures." For the two heavier loads, one test section exhibited significant present serviceability index decreases during the springs of each test year and was classified a "borderline" performer. Again, for all three loads one test section displayed no changes in present serviceability index or cracking. These sections were classified "thinnest survivors." It was hypothesized that since performance characteristics depicted by these curves were so completely different that the radial stresses and strains and the vertical strains in the subgrade would be different.

ANALYSIS

The input data available for stress and strain computations were as follows:

(a) complex modulus values as a function of termperature and rate of loading for the asphalt concrete surfacing (determined by Coffman).[10]

(b) complex modulus values for the asphalt concrete base as a function of temperature and rate of loading (determined in The Asphalt Institute laboratory).[11]

(c) deflection data for all periods of the year.

(d) pavement temperature data measured for the top four inches of asphalt concrete at hourly intervals during the entire testing period.

For complex modulus determinations a frequency loading of one cycle per second was chosen for use with deflection measurements. Coffman[10] describes the rationale for picking this loading frequency. Temperature data for modulus determinations were obtained from the AASHO Road Test data system 3300 for the time of deflection measurement. Deflection times were estimated from the operation schedule and the temperatures recorded with the deflection data system. Where temperatures were required at depths greater than four inches, the Southgate[12] approach was used to estimate these temperatures. The following equations were used to determine stiffness for the asphalt concrete surface and base:

Surface:

$$\log E = 6.56495 - .01178(T)^{.995}$$ Eq. 1 (Coffman data[10])

Base:

$$\log E = 6.32456 - .000012(T)^{2.51}$$ Eq. 2 (Kallas[11])

where E = modulus at 1 cps
T = temperature F.°

No test data were available to estimate the subgrade and subbase modulus values directly. Even if such values had been available it would have been difficult to estimate the condition of the base and subgrade for each of the 315 deflection measurments analyzed. Therefore, subgrade and subbase modulus values were estimated from the pavement deflection measurement using Kirk's[13] simple formula for deflection. Deflection data were selected to represent all seasons of the year and all loads, as shown by Figures 3 and 4. These represent approximately 50% of all deflections taken on the bituminous base sections. A modular ratio of subbase to subgrade of 2:1 was assumed from the work of Dormon.[14] Once the modulus values for the asphalt granular subbase and subgrade were known, it was possible with an assumed Poisson's ratio to compute the

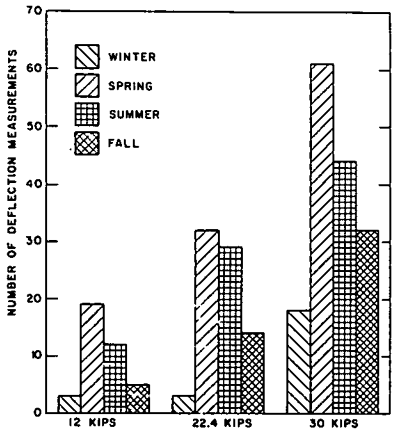

Fig. 3 DATA INPUT BY SEASON

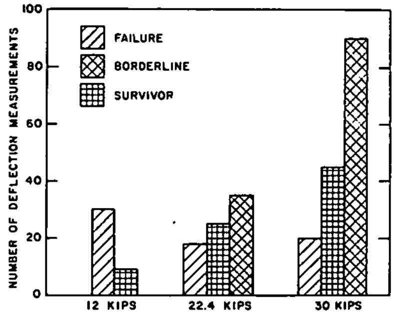

Fig. 4 DATA INPUT BY PERFORMANCE CLASSIFICATION.

strains, stresses and deflections using the Chevron elastic layer computer program. A single plate loading configuration was assumed to represent the dual wheel configuration. Hence the maximum horizontal tensile strains and stresses are radial strains and stresses. Poisson's ratios of 0.45 and 0.4 were assumed for asphalt concrete temperatures greater than and less than 70°F. A check on the computations was made by comparing computed deflections to those measured. The computed values were in all cases within .002" of those measured.

In order for the computed stresses and strains to be applied to a repeated load analysis, the performance classifications had to be identified with numbers of load repetitions. Pavements failing the first spring, "failures," carried a total of 121,000 load repetitions on the average. Of these only 57,000 were applied with no frost in the subgrade. For the "borderline" test sections, repetitions to failure were harder to define. Test sections carrying the 22,400 lb. axle load did not reach the terminal present serviceability index of 1.5 by the end of the test. Extrapolating the serviceability index trend it was estimated that 2.03 million load repetitions would have been required to reduce the serviceability index to 2.5. Test sections carrying the 30,000 lb. single axle load failed at an average of 644,200 load repetitions. A logarithmic average between the 22,400 and 30,000 single axle load results would fall close to the total number of load repetitions imposed on the test sections, 1.12 million. Therefore, this class was identified with 1.12 million load repetitions. Load repetitions to failure for the thinnest surviving test sections could not be determined.

HORIZONTAL TENSILE STRAIN (Radial Strain)

The radial strain on the bottom of the asphalt concrete determined at the time of each deflection measurement was plotted against the modulus of the asphalt concrete. Since the modulus was not a constant with depth the modulus for the bottom of the base layer was used. Figures 5 and 6 show the strain as a function of modulus on log-log scales. The top curve of Figure 5 shows the data by performance classification. The remaining curve in Figure 5 and those in Figure 6 show the data breakdown by load for each performance classification. Several conclusions can be drawn from these curves. First, the data for the three performance classifications are indeed completely separate as hypothesized. Therefore, it seems reasonable that these curves are indeed a family of curves which can be identified with two levels of load repetitions. Secondly, they do suggest that an asphalt concrete pavement can withstand greater strains when its stiffness is lowered.

The data plotted in Figures 5 and 6 excluded all data points for periods when measurable depths of frost were recorded in the subgrade. Figure 7 shows the computed curves from Figures 5 and 6 together with the "frozen" data. The data plot significantly below the curves suggesting that these points are in reality on other curves representing a much larger number of load repetitions.

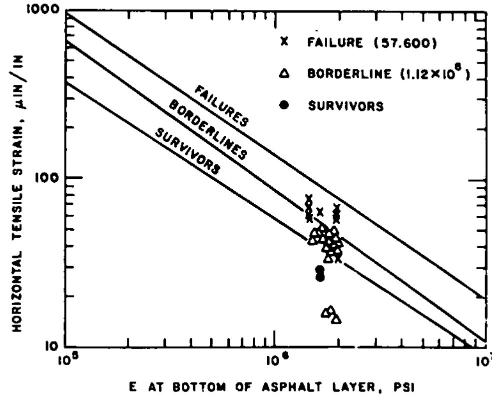

Fig. 7 COMPARISON WITH DATA FROM FROZEN PERIODS.

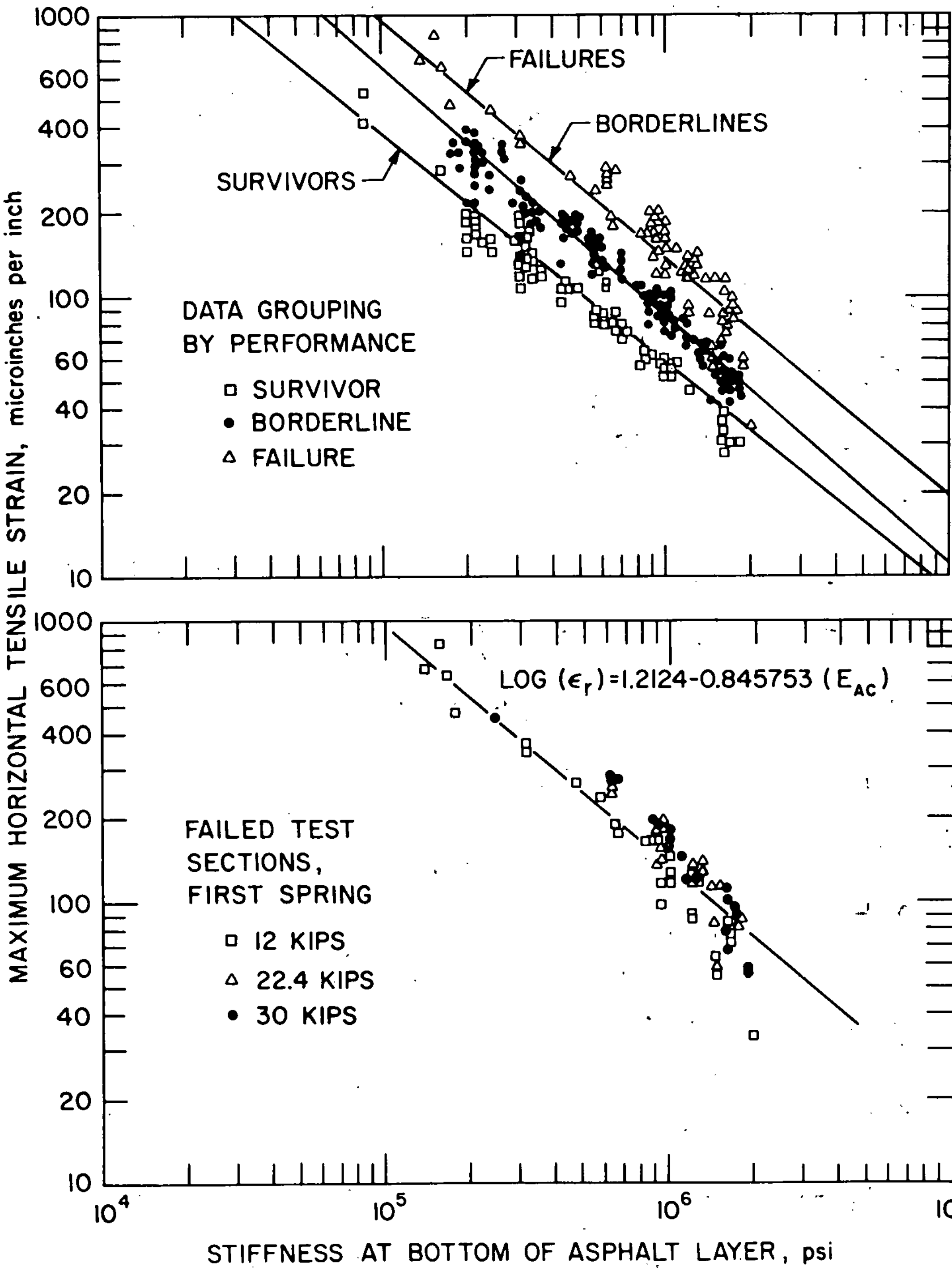

Fig. 5 STRAIN LEVELS BY PERFORMANCE CLASSIFICATION.

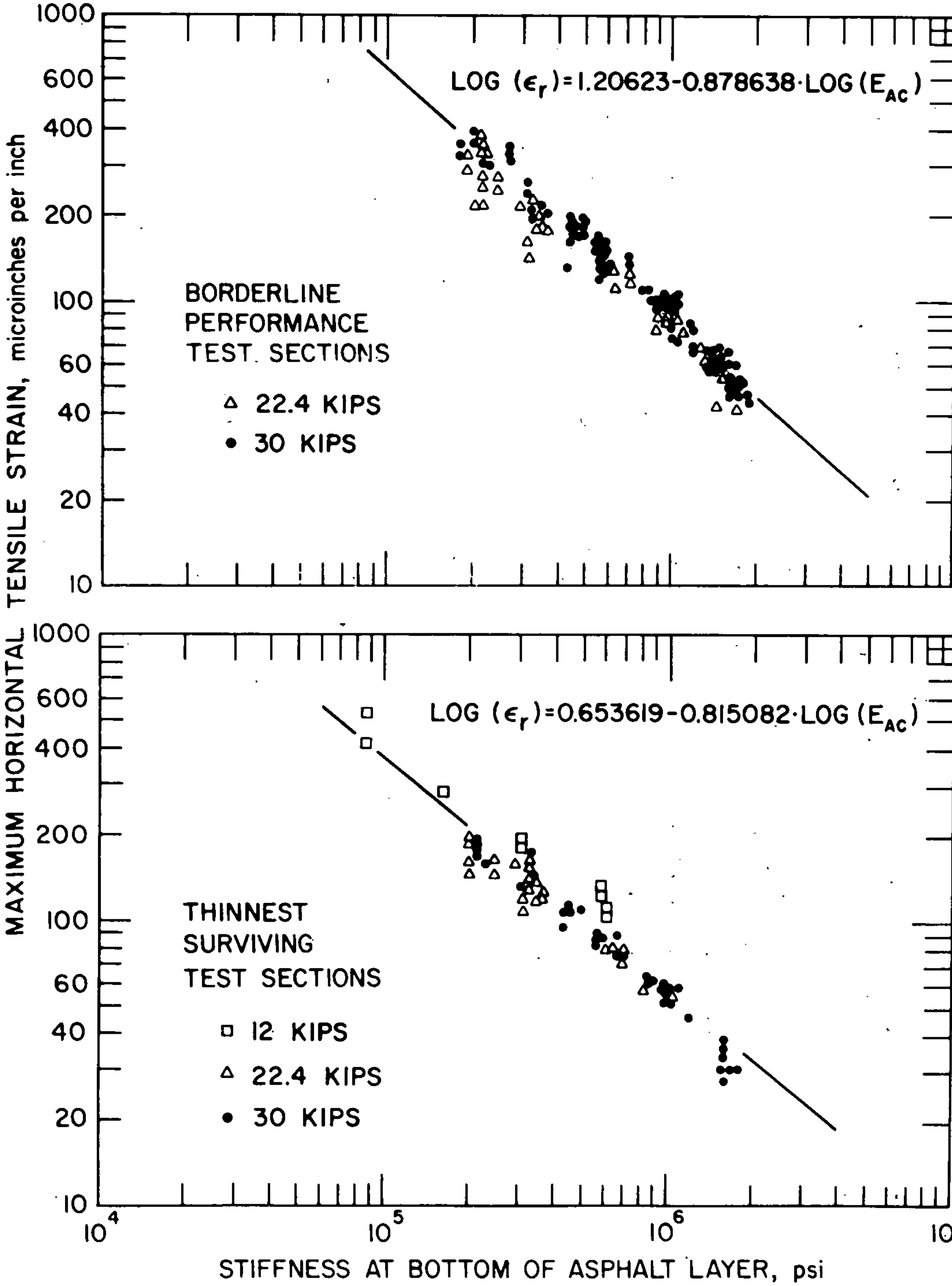

Fig. 6 STRAIN LEVELS BY PERFORMANCE CLASSIFICATION.

The plot of radial strain versus stiffness modulus can be resolved into a plot of load repetition curves showing radial strain as a function of load applications to failure. The regression analyses for two sets of curves are shown in Figure 8. To construct

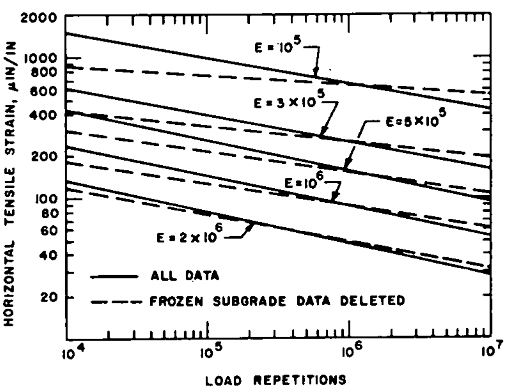

Fig. 8 LOAD REPETITION FAILURE CURVES WITH AND WITHOUT DATA FROM FROZEN PERIODS.

the curves it was necessary to assume that the relationships were linear in the log-log transformation. The solid lines represent the result from analyzing all deflection data (for frozen and unfrozen subgrades). Test sections classified as "failures" were identified as having 121,000 load repetitions to failure. The dashed lines are those determined with all strain data and load repetitions associated with the frozen periods deleted. "Failures" for this analysis were associated with 57,000 load repetitions. For both solid and dashed lines 1.12×10^6 load repetitions to failure were identified with the borderline performance test sections. Equations describing the two sets of curves are given below.

For all data:

$$\log e = 1.2458 - .67296 \log E - .0065461 \log N_f - .034001 \log E \log N_f$$

For deletion of data associated with frozen subgrades:

$$\log e = -1.34114 - .28646 \log E + .403893 \log N_f - .094801 \log E \log N_f$$

The solid lines are nearly parallel and have similar slopes to laboratory fatigue curves,[15] whereas the dashed lines are slightly flatter than laboratory fatigue curves. The selection of design curves from this analysis is a matter of judgment. Witczak,[16] in his paper to this conference, chose the solid curves because of their nearly parallel nature and the support from laboratory data for the slopes. As an aid to selection for design, the data scatter represented by the root mean square error is given in Table 1.

Table 1

HORIZONTAL TENSILE STRAIN FAILURE CRITERIA FOR LOG STRAIN = INTERCEPT + SLOPE (LOG E)

Performance Classification	Regression Results Intercept.	Slope	RMSE
All Data			
Failures	1.2124	-.84575	.11600
Borderlines	1.20623	-.87864	.10862
Survivors	.653619	-.81508	.12914
Frozen Subgrade Data Deleted			
Failures	.57972	-.73732	.13007
Borderlines	1.1021	-.85993	.11058
Survivors	.53188	-.79308	.12894

In translating the strain - stiffness curves to strain - load repetition curves, it is assumed that if one were to test at a given stiffness level then a unique number of load repetitions would occur for a given strain. It is possible that a data bias exists because of the single environmental locality of the AASHO Road Test. To check on the possibility of such a data bias an analysis was made to develop the load repetition curves using Miner's hypothesis and the actual distribution of load applications as a function of temperature. Details of this analysis are given in the Appendix. The result was very minor changes in the load repetition curves as shown in Figure 9, which were judged to be insignificant.

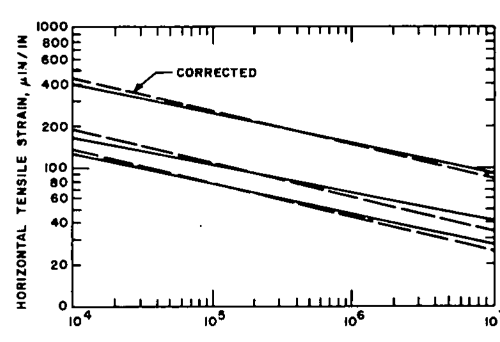

Fig. 9 COMPARISON OF FAILURE CRITERIA CURVES WHEN CORRECTED FOR ENVIRONMENTAL DATA BIAS.

An application of the load repetition curves to a 5" Full-Depth asphalt pavement carrying an 18-kip axle load for subgrade modulus values of 6000 and 15,000 psi is given in Figure 10. The plotted points are strains determined in an elastic layered analysis. These strains when plotted on the load repetition curves show that the higher asphalt layer stiffnesses are more damaging than the lower asphalt stiffnesses which is in agreement with the philosophy of some highway department engineers.

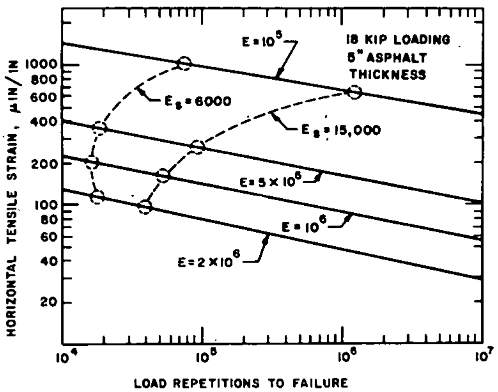

Fig. 10 COMPARISON OF DAMAGE WITH CHANGES IN ASPHALT STIFFNESS.

HORIZONTAL TENSILE STRESS (Radial Stress)

In the same manner that strain was plotted against stiffness of the asphalt layer, radial stress was also plotted. At first glance the data appeared to be much more scattered than those for radial strain. Further examination indicated that the subgrade modulus had a large effect on the data scatter. When subgrade modulus was recognized the expected curve of increasing stress with increasing stiffness was found, as shown in Figure 11. Also load differences appeared to explain some of the scatter. These plots show that a failure stress needs to be qualified by the subgrade modulus it pertains to. Failure stresses increase with decreasing subgrade modulus. Hence this design criterion is not entirely independent of other structural considerations, as was the radial strain.

Radial strain as a function of load repetitions to failure for various levels of asphalt and subgrade modulus values are given in Figure 12. Regression analysis results for the plotted curves in Figures 11 and 12 are given in Table 2. At the time of writing these criteria have not been applied to airfield or highway design.

VERTICAL STRAINS

Vertical strains were analyzed in the same manner as for radial strains and stresses. The changes in vertical strain with asphalt concrete modulus were considerable. Since rutting was not an observed failure mechanism except in instances where cracking occurred initially, the values of vertical strain at various levels of load

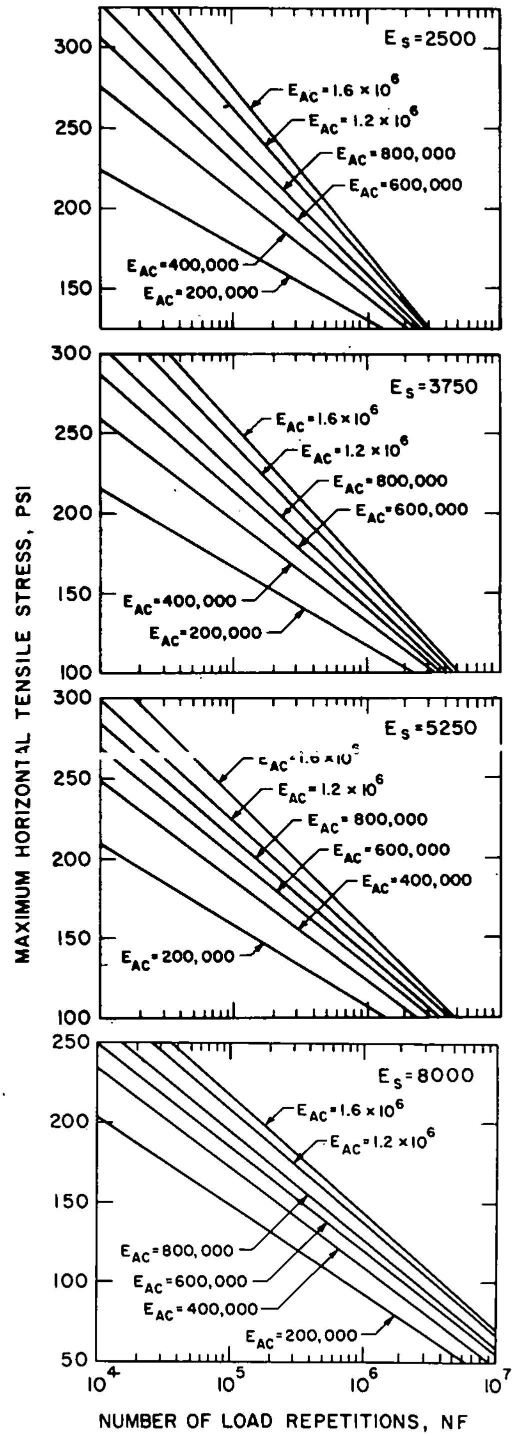

Fig. 12 LOAD REPETITION CURVES FOR HORIZONTAL STRESS CRITERIA.

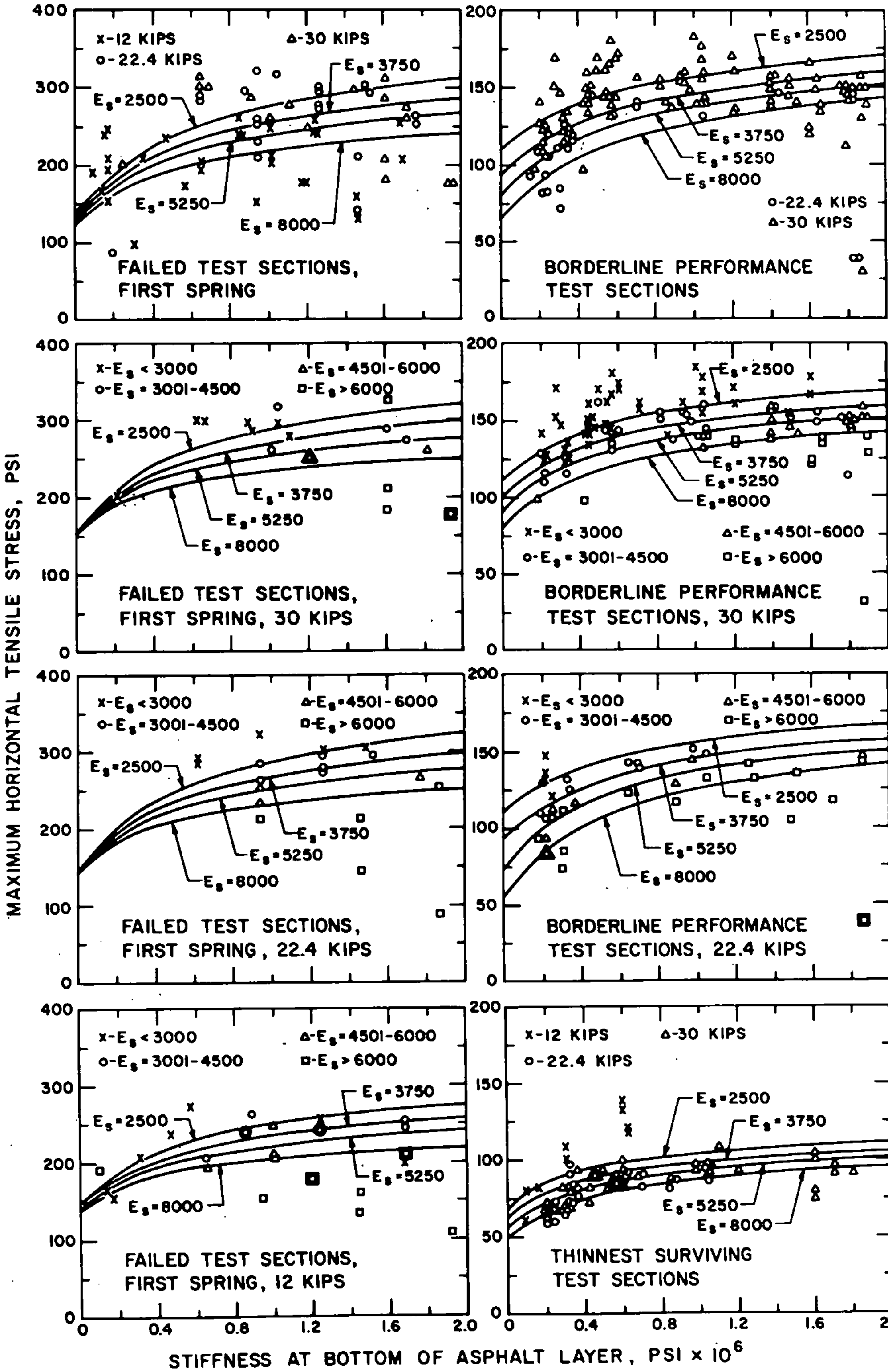

Fig. II STRESS LEVELS BY PERFORMANCE CLASSIFICATION.

Table 2

HORIZONTAL TENSILE STRESS FAILURE CRITERIA
REGRESSION ANALYSIS RESULTS*

Regression Results	Performance	Classification
Data For Frozen Subgrade Deleted	Failures (57,000 reps)	Borderline (1.12×10^6 reps)
A_0	-1878.94	590.67
A_1	422.91	-37.81
A_2	412.74	-203.70
A_3	-87.53	23.86
R^2	.885	.874
RMSE	25.1 psi	10.4 psi

$$*\text{Stress} = A_0 + A_1 \log E_b + A_2 \log E_s + A_3 (\log E_b)(\log E_s)$$

where

b = base
s = subgrade

repetition were only of interest to compare with other criteria. Table 3 presents the vertical strain results from regression analyses associated with each performance classification. Design criteria published by Dormon and Metcalf[5] are compared. The only valid comparisons are those where pavements survived a significant number of load repetitions when pavement modulus values were at 200,000 psi or lower. This excludes the failure classification since these test sections failed before the first summer of test traffic. From the valid comparison it is obvious that vertical strains in the subgrade exceeded those recommended for design by Dormon and Metcalf. Since little or no rutting was measured in the subgrade soil it is the opinion of this author that the Dormon and Edwards vertical strain criteria are conservative.

An interesting conclusion can be drawn from the subgrade vertical strain and the associated measured rut depths given in Table 3. As a function of load the rut increased from the lightest to the heaviest load for the thinnest surviving pavements. Vertical strains in the subgrade, however, decreased with increases in rut depth. Each survivor test section had 1.12×10^6 load repetitions applied to it. Hence the magnitude of vertical strain in the top of the subgrade does not explain the surface rutting observed. Other studies conducted at the Road Test[9] indicate that less than 10% of the surface rut reflected into the subgrade. Since increasing base thickness

Table 3

SUMMARY OF VERTICAL STRAIN ANALYSIS

Classification No. of Load Repetitions to Failure	Axle Load Kips	Regression Equations* Slope	Intercept	R^2	MSE	Shell Vertical Strain Criteria**	AASHO Computed Strain Values** Ave.	Ave. -2 RMSE	Measured Rut Depth (Inches)
FAILURES (57,400 load repetitions)	12	-1.07818	3.04914	.899784	.10417	1120	2400	543.6	
	22.4	-1.14834	3.61938	.625782	.125764		3700	721.4	
	30.0	-1.15021	3.61388	.825974	.117772		3700	760.7	
BORDERLINE (1.12×10^6 load repetitions)	22.4	-.808804	1.20951	.927274	.0711949	630	830	243.3	
	30.0	-.796925	1.24201	.926375	.0674619		1040	315.8	
SURVIVORS (unknown load repetitions)	12	-.730062	.735	.56954	.17563	Unknown	730	105.8	.25
	22.4	-.615682	-.0561679	.905311	.0463689		480	178.8	.38
	30.0	-.690541	.377486	.92566	.0573453		520	173.2	.50

*Model $\log e_v = a_0 + a_1 \log E$
**Values computed for a pavement modulus value of 200,000 psi

above the minimum required to prevent cracking did not reduce the rutting, it is apparent that limiting the subgrade vertical strain would not prevent rutting for the Road Test asphalt base pavements. Hence it is concluded that limiting the subgrade vertical strain in design may prevent rutting in the subgrade but it will not insure the prevention of rutting at the pavement surface.

SUMMARY AND CONCLUSIONS

1. Horizontal tensile strains and stresses at the bottom of the asphalt base and vertical strains in the top of the subgrade were correlated to performance trends at the AASHO Road Test. From such a correlation, levels of radial strain, radial stress and vertical strain in the subgrade could be identified with unique numbers of load repetitions.

2. Stiffness of the asphalt concrete provided a strong influence on critical strains or stresses. By comparison with limiting horizontal tensile strain values published in the literature it is apparent that asphalt pavements can tolerate higher strains at lower stiffnesses.

3. Horizontal tensile strains at failure for given numbers of load repetitions to failure were independent of load effects and hence appeared to be entirely a function of the asphalt mix properties.

4. Horizontal tensile stresses at given load repetitions to failure were highly dependent on subgrade stiffness and hence more difficult to use as design criteria than strain.

5. Vertical strains in the top of the subgrade did not correlate to the measured rut depths. Therefore, limiting vertical strain in the subgrade may prevent deformation in the subgrade but it will not insure the prevention of rutting at the pavement surface.

APPENDIX

DERIVATION OF STRAIN-LOAD REPETITION CURVES

It is possible to derive a family of strain-load repetition curves as a function of asphalt stiffness that considers the distribution of loads with stiffness experienced at the AASHO Road Test. The resulting curves should be free of any environmental bias that imposes itself by virtue of the data base being entirely from the AASHO Road Test. In Figure A-1, the curve for 121,000 load repetitions is drawn from Figure 5 for the failed test sections. The curve designated 626,000 load repetitions represents the average life of borderline performance test sections from the 30-kip axle load test. For the third curve, the

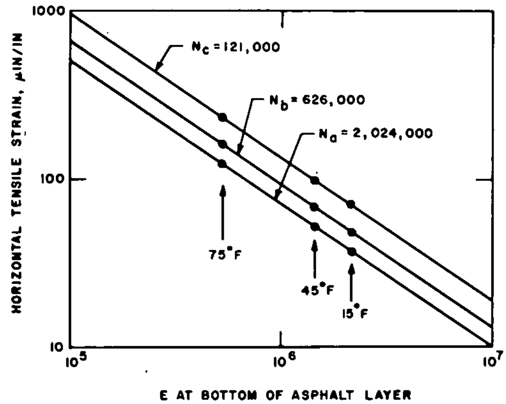

Fig. A-1 STRAIN-E DATA FOR ANALYSIS INPUT.

22.4 kip axle load test sections were assigned a value of 2.024 x 10^6 load test repetitions based on an extrapolation of their present serviceability index trends. By dividing the temperature range of 0° to 90°F into three equal parts and converting to stiffness of the asphalt concrete for 1 cycle per second loading time, three values of strain are identified for each curve, or nine values in total. For each strain and stiffness pair there exists an actual number of load repetitions applied. These are known from the loading history and are given in Table A-1.

The load repetition values were derived from the histograms given in Figures A-2 and A-3. These in turn were derived from monthly average pavement temperature - air

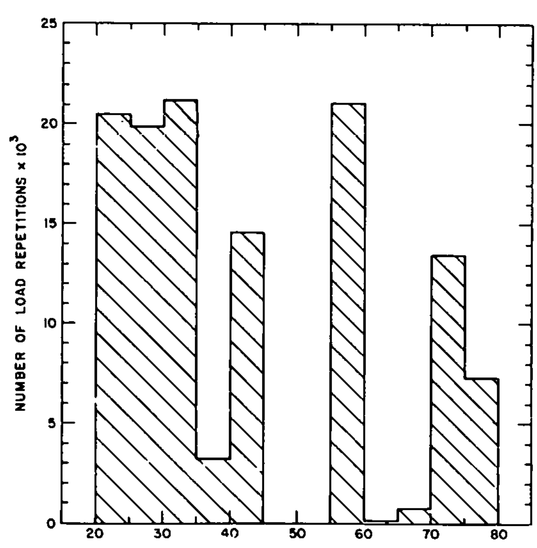

Fig. A-2 FAILURES

Table A-1

SUMMARY OF ANALYSIS INPUT DATA

Strain - Load Repetitions Pairs μ in/in - No.	Pavement E_1 530,000 psi	Pavement E_2 1,428,000 psi	Pavement E_3 2,095,000 psi	Total Load Repetitions
e_{ia}	122.5	52.3	37.5	
n_{ia}	1,021,495	783,213	219,580	2,024,000
e_{ib}	160.0	68.2	49.0	
n_{ib}	198,000	289,000	139,000	626 000
e_{ic}	237.5	100.1	76.6	
n_{ic}	26,199	52,222	52,616	121,037

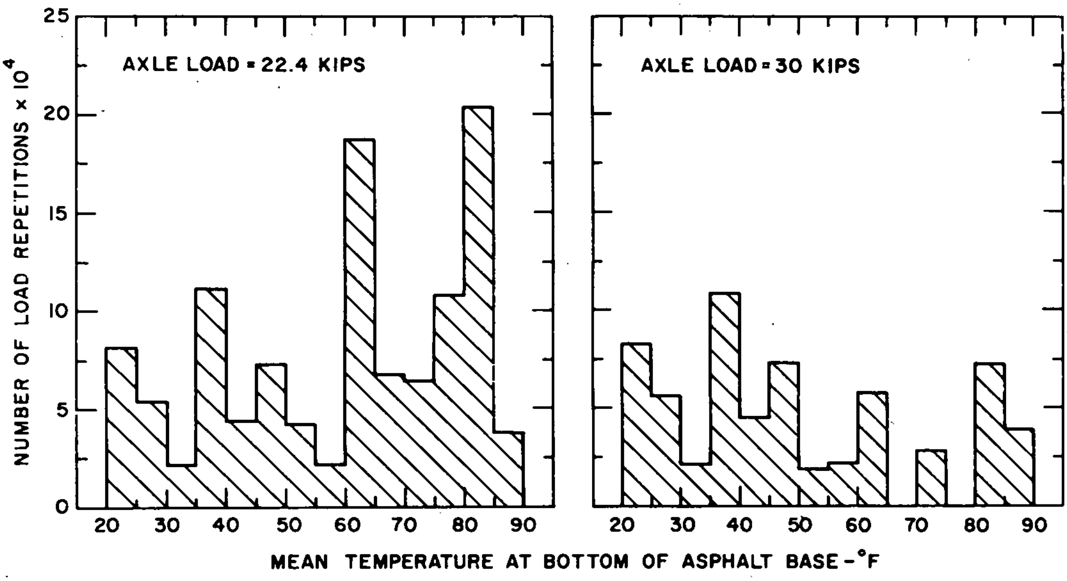

Fig. A-3 BORDERLINE SECTIONS.

temperature relationships developed by Witczak.[18]

Using Miner's hypothesis, <u>repetitions to failure</u> for each strain - stiffness pair was determined in the following manner:

Let n_{ij} be the actual number of weighted repetitions occurring during an average temperature range (associated with $\bar{E}_i$ for the j^{th} failure curves

$i = 1, 2, 3$
$j = a, b, c$

Let e_{ij} be the maximum tensile strain at the bottom of the A.C. layer associated with the i^{th} and j^{th} point on the failure curves

$n_{1a} + n_{2a} + n_{3a} = N_a = 2,024,000$
$n_{1b} + n_{2b} + n_{3b} = N_b = 626,000$
$n_{1c} + n_{2c} + n_{3c} = N_c = 121,000$

Using Miner's hypothesis of damage accumulation and letting

N_{f1a} = allowable rep. to failure that e_{1a} will cause at E_1 for N_a curve

N_{f1b} = allowable rep. to failure that e_{1b} will cause at E_1 for N_b curve

N_{f1c} = allowable rep. to failure that e_{1c} will cause at E_1 for N_c curve

N_{f2a} = allowable rep. to failure that e_{2a} will cause at E_2 for N_a curve

N_{f2b} = allowable rep. to failure that e_{2b} will cause at E_2 for N_b curve

N_{f2c} = allowable rep. to failure that e_{2c} will cause at E_2 for N_c curve

N_{f3a} = allowable rep. to failure that e_{3a} will cause at E_3 for N_a curve

N_{f3b} = allowable rep. to failure that e_{3b} will cause at E_3 for N_b curve

N_{f3c} = allowable rep. to failure that e_{3c} will cause at E_3 for N_c curve

$$\therefore \frac{n_{1a}}{N_{f1a}} + \frac{n_{2a}}{N_{f2a}} + \frac{n_{3a}}{N_{f3a}} = 1 \qquad \text{(Equation 1)}$$

$$\frac{n_{1b}}{N_{f1b}} + \frac{n_{2b}}{N_{f2b}} + \frac{n_{3b}}{N_{f3b}} = 1 \qquad \text{(Equation 2)}$$

$$\frac{n_{1c}}{N_{f1c}} + \frac{n_{2c}}{N_{f2c}} + \frac{n_{3c}}{N_{f3c}} = 1 \qquad \text{(Equation 3)}$$

Assuming that the true family of fatigue curves (N_f - e) are parallel, i.e., $c_1 = c_2 = c_3$ for various temperatures (stiffnesses E_1, E_2, E_3) then the following are true

$$N_{f1a} = K_1\left(\frac{1}{e_{1a}}\right)^c \qquad N_{f1b} = K_1\left(\frac{1}{e_{1b}}\right)^c$$

$$N_{f1c} = K_1\left(\frac{1}{e_{1c}}\right)^c$$

$$N_{f2a} = K_2\left(\frac{1}{e_{2a}}\right)^c \qquad N_{f2b} = K_2\left(\frac{1}{e_{2b}}\right)^c$$

$$N_{f2c} = K_2\left(\frac{1}{e_{2c}}\right)^c$$

$$N_{f3a} = K_3\left(\frac{1}{e_{3a}}\right)^c \qquad N_{f3b} = K_3\left(\frac{1}{e_{3b}}\right)^c$$

$$N_{f3c} = K_3\left(\frac{1}{e_{3c}}\right)^c$$

or $$\frac{N_{f1a}}{N_{f1b}} = \frac{(e_{1b})^c}{(e_{1a})^c} \quad \therefore \quad N_{f1a} = N_{f1b}\left(\frac{e_{1b}}{e_{1a}}\right)^c$$

(Equation 4)

$$\frac{N_{f1a}}{N_{f1c}} = \frac{(e_{1c})^c}{(e_{1a})^c} \quad \therefore \quad N_{f1a} = N_{f1c}\left(\frac{e_{1c}}{e_{1a}}\right)^c$$

(Equation 5)

$$\frac{N_{f2a}}{N_{f2b}} = \frac{(e_{2b})^c}{(e_{2a})^c} \quad \therefore \quad N_{f2a} = N_{f2b}\left(\frac{e_{2b}}{e_{2a}}\right)^c$$

(Equation 6)

$$\frac{N_{f2a}}{N_{f2c}} = \frac{(e_{2c})^c}{(e_{2a})} \quad \therefore \quad N_{f2a} = N_{f2c}\left(\frac{e_{2c}}{e_{2a}}\right)^c$$

(Equation 7)

$$\frac{N_{f3a}}{N_{f3b}} = \frac{(e_{3b})^c}{(e_{3a})^c} \quad \therefore \quad N_{f3a} = N_{f3b}\left(\frac{e_{3b}}{e_{3a}}\right)^c$$

(Equation 8)

$$\frac{N_{f3a}}{N_{f3c}} = \frac{(e_{3c})^c}{(e_{3a})} \quad \therefore \quad N_{f3a} = N_{f3c}\left(\frac{e_{3c}}{e_{3a}}\right)^c$$

(Equation 9)

The nine equations above allow the solution of the N_{fij} values which are the desired load repetitions to failure. Analyses for chosen values of C from 4 to 6 revealed that sets of positive N_{fij} values could only be determined for values of C from 4.20 to 4.23. Assuming a mid-value of 4.215 the solution is given in Table A-2.

These data are the basis of the "corrected curves" shown in Figure 9. As concluded in the main section of the paper there would appear to be little or no environmental bias in the development of the failure criteria.

Table A-2

SUMMARY OF LOAD REPETITIONS TO FAILURE DETERMINED FROM ANALYSIS

Stiffness, psi	Load Repetitions to Failure at Given Strain, μ in/in.					
	N_{f1j}	e_{1j}	N_{f2j}	e_{2j}	N_{f3j}	e_{3j}
2,095,000	131,022	122.2	694,482	52.3	2,130,078	37.5
1,428,000	122,908	160.0	651,475	68.2	1,998,830	49.0
530,000	104,858	237.5	555,801	100.1	1,705,290	76.6

REFERENCES

1. Wilkins, E. B., Outline of a Proposed Management System, Proceedings, 1968 Convention of the Canadian Good Roads Association.

2. Haas, R. C. G., Terrel, R. L., and Kasianchuk, D. A., Pavement Design Subsystems for Fatigue, Permanent Deformation and Shrinkage Cracking, Paper offered to the Third International Conference on the Structural Design of Asphalt Pavements, London, 1972.

3. Michelow, J., Warren, H. and Dieckmann, W. L., Numerical Computation of Stresses and Strains in a Multiple-Layered Asphalt Pavement System, California Research Corporation, Richmond, California, September 24, 1963.

4. Hicks, R. G. and Finn, F. N., Analysis of Results from the Dynamic Measurements Program on the San Diego Test Road, Proceedings, AAPT, 1969.

5. Dormon, G. M. and Metcalf, C. T., Design Curves for Flexible Pavements Based on Layered System Theory, U.S. Highway Research Board Record No. 71, 1965.

6. Deacon, J. A. and Monismith, C. L., Laboratory Flexural Fatigue Testing of Asphalt Concrete with Emphasis on Compound Loading, Research Record 158, Highway Research Board, 1967.

7. Pell, P. S. and Taylor, I. F., Asphaltic Road Materials in Fatigue, Proceedings, AAPT 1969.

8. Kingham, R. I., Laboratory Fatigue and Its Relationship to Pavement Performance, Paper offered to the Third International Conference on the Structural Design of Asphalt Pavements, London, 1972.

9. The AASHO Road Test, Report 5, Pavement Research, HRB Special Report 61E, 1962.

10. Coffman, B. S., Kraft, D. C., and Tamayo, J., A Comparison of Calculated and Measured Deflections for the AASHO Road Test, Proceedings, AAPT, 1964.

11. Kallas, B. F., and Riley, J. C., Mechanical Properties of Asphalt Pavement Materials, Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1967.

12. Southgate, H. F., An Evaluation of Temperature Distribution Within Asphalt Pavements and Its Relationship to Pavement Deflection, Kentucky Department of Highways, Research Report, KYHPR-64-20, April 1968.

13. Kirk, J. M., Calculating the Thickness of Road Courses, Proceedings, 12th Congress of the Permanent International Association of Road Congresses, Rome, 1964.

14. Klomp, A. G. J., and Dormon, G. M., Stress Distribution and Dynamic Testing in Relation to Road Design, Proceedings, Australian Road Research Board, Vol. 2, Part 2, 1964.

15. Epps, J., Fatigue of Asphalt Concrete Mixtures -- Summary of Existing Information, ASTM Symposium, Atlantic City, 1971.

16. Witczak, M. W., Documentation of The Asphalt Institute's Design Method for Full-Depth Asphalt Airfield Pavements, a paper offered to the Third International Conference on the Structural Design of Asphalt Pavements, London, 1972.

Summary Discussion

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At the Symposium on the Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Failure, we were impressed with the advances that have been made in the understanding of pavement failure and in our ability to predict that failure with respect to the fatigue phenomenon. The other modes of failure, rutting and roughness, have also been covered to some extent by several of the authors, especially Havens and Deen. However, one of the big unanswered problems has to do with the interaction of those failure modes and its effect on pavement life. We are currently able, in a rough way, to predict fatigue performance and rutting performance, but as yet we are unable to combine the two with much certainty or to actually include the roughness effect as a performance modifier.

Barksdale and Hicks listed the many factors inherent in the design of pavements: environment, traffic, material properties, construction variables, maintenance variables, and economics. Because of the many factors and the many variables associated with those factors, we at the Federal Highway Administration are striving toward a probabilistic or stochastic approach to design. Barksdale and Hicks also pointed out that the characterization of the materials, especially granular materials, is not a straightforward linear problem but that moduli and Poisson's ratios vary with the stress state. That is true; however, it is yet to be shown that this effect is important enough to warrant complicating the testing and analysis procedures to account for it. Stress was also placed on the effects of moisture and temperature on pavement performance. Nearly everyone working on new methodology for flexible pavement design seems to be confident that the temperature variations can be accounted for, but so far I have heard little if any assurance that moisture variations can be as easily handled.

In their discussion of layered systems analysis, Barksdale and Hicks presented a rather detailed outline of the history of what might be termed "The Struggle to Develop a Rational Design Method for Flexible Pavements." I first entered into that struggle in 1946 as Don Burmister's assistant at Columbia University—not that I played a great part, but I was exposed to it. Later at Purdue University I spent about 2 years measuring and analyzing stresses and strains in layered systems. As you can see, this struggle has been going on for at least 25 years, and only now are we beginning to see what could be a "negotiated peace."

Pell performed an excellent job of defining the technical terms and criteria used in the fatigue analysis of pavement performance. He also related the problems associated with fatigue testing and fatigue characterization of the pavement components. According to him, one of the major problems associated with the testing is that of duplication of in situ stress states in laboratory test procedures. Here again, we all agree that it is a problem, but we differ as to how much of a problem it is. The situation is still in such flux that I can continue to hope that the problem is negligible. I am convinced that if Majidzadeh continues to make progress in his work in fracture mechanics the problem will disappear. In fact, he states that in his work the prediction of fatigue life of pavements from laboratory tests is independent of the mode of loading or the specimen geometry.

Pell also stated that the slope of the fatigue line (fatigue characteristic) appears to depend on the stiffness characteristics of the mix and the nature of the binder. He said further that the 2 factors that appear to be of primary importance are binder content and voids content with an existing optimum fatigue life dependent on the relations of binder, filler, and voids. For good fatigue performance of thick asphalt construction, a mix of maximum stiffness should be the objective, and the quantities of filler and binder should be such that a condition of maximum tensile stiffness associated with minimum voids is produced.

Hudson provided a great deal of information concerning input variables, how they may or should be obtained, and how they may be used in the design system. One aspect of fatigue that has not been extensively discussed is that of thermal fatigue. Some excellent work in that area has been done in Texas.

Deacon outlined a design process that is basically a trial-and-error procedure wherein (a) a trial structure is assumed; (b) the structure is analyzed by estimating the levels of the critical stresses and strains anticipated under in-service loading; (c) the structure is evaluated by comparing the estimated stresses and strains with tolerable levels derived from failure criteria; and (d) modifications are made to the structure as necessary, and the process is repeated until a satisfactory design has evolved. He defined failure, as do most of the authors, but there is still much room for agreement among them.

Finn reminded us that we must determine how much cracking is bad, what kinds of cracking are bad, and what happens to a pavement after cracking occurs. Those are all questions that need to be resolved in order for cracking predictions to be meaningful. A certain amount of cracking is acceptable in terms of riding quality, but so far we are not sure what it means in terms of structural integrity. Finn discussed possible criteria for using cracking as a performance parameter and suggested that subjective evaluations of the future utility of a given pavement together with objective measurements of the amount of cracking existing on the pavement be used to develop useful cracking criteria.

Terrel gave excellent reviews of several of the important field and test track studies. He has done some fine work with his test track and has provided much data that have been used in comparison with predicted results. He concluded by saying, "It would appear that prediction of fatigue failure in actual pavements is feasible. However, there appears to be a basic lack of knowledge in the actual behavior of pavements under varying loads and environments."

Witczak presented an excellent example of the use of the latest available methodology in the development of an actual design procedure for airfield pavements. When a truly rational design method is established, we should be able to leave off the labels of airport or highway and say we have procedures for the design of any pavement for any purpose.

At FHWA, we are particularly proud of the presentation by Havens and Deen. It is a good example of the work of the personnel of a state highway department research group. It typifies the attitude of many of the state departments toward research and their efforts to take advantage of every opportunity to improve their operations. Although it cannot be labeled as truly rational, it is a major step forward.

Most of the presentations at the symposium were directly or indirectly associated with the FHWA research project on the rational design of flexible pavements. That project originated in 1965 and is currently carried as Project 5C, New Methodology for Flexible Pavement Design. Administrative funding of this project will end with fiscal year 1973 funding; however, in-house and HPR portions of the project will continue for several more years.

Recently an administrative fund contract that was concerned with the subject of this symposium was completed. The results of that study will be reported by Smith and Nair in a paper to be published in the 1973 Highway Research Record series. The authors report the following conclusions.

1. The predominant parameter variation causing uncertainty in fatigue life is that associated with construction control of air voids in the asphalt concrete.
2. A more accurate definition of the fatigue failure criteria for asphalt concrete would be more beneficial than improvement of the constitutive material characterization beyond isotropic linear elasticity.
3. The uncertainty associated with characterization of pavement materials by isotropic linear elasticity can contribute significant uncertainty to the prediction of fatigue life. However, that induced uncertainty in fatigue life is of less significance than the uncertainty induced by field control of air voids or fatigue criteria definition.
    a. Improved characterization of asphalt concrete would be most beneficial for full-depth and thick asphalt concrete surface pavements.
    b. More accurate characterization of untreated granular base course material would also be advantageous, especially for pavements located in hot climates.
    c. An isotropic linear elastic characterization of subgrades is adequate if the characterization is performed under levels representative of in-service subgrades.
4. Consideration of variations in water content and densities of in situ base course and subgrade materials would be beneficial for pavements located in hot climates.
5. Within the existing techniques for considering temperature in the analysis, the ability to predict the temperature is sufficiently accurate.
6. Thickness control of the asphalt concrete layer is now sufficiently accurate.

It should be recognized that the adequacy of a material characterization is a dynamic phenomenon. As our ability to describe and control the effects of other parameters influencing the fatigue life of flexible pavements improves, characterizations that are now adequate may become inadequate.

At a meeting at the University of Nottingham in September 1972, I announced that the FHWA would have a flexible pavement design system developed and ready for use by fiscal year 1975. Many of those attending that meeting intimated that this was an impossible goal. However, I still stick by that estimate even though Bill Kenis says it is his neck I am sticking out.

Many of us in the FHWA Offices of Research and Development are essentially research managers, although we do have a very active in-house program. Our superiors insist that we produce a useful product in the shortest time and at the least cost possible. It is my belief that the only way to produce a rational design method is to assemble all available knowledge into the most logical and feasible system and give it a trial. Such trials will soon point out whether we have a good system or whether our information is deficient and more research is needed. Therefore, we are progressing along those lines.

Briefly, our plan is to assemble together in a modular system all of the subsystems we have or will have when our staff research and contract effort is completed. We will use much of the information presented at this symposium. The fatigue concept will be a modular subsystem with a fracture mechanics module as an alternative; elastic and viscoelastic stress analysis and strain computation modules will be included. We will then try those alternatives in actual designs and compare them in highway department designs, test tracks, and other accelerated test facilities. In fact, we have already started that program at Pennsylvania State University. The university, the Pennsylvania Department of Transportation, The Asphalt Institute, the Crushed Stone Association, and FHWA are cooperating in an evaluation of some of the existing methodology. We are particularly interested in evaluating the M.I.T. viscoelastic methods for predicting rutting. The materials will be tested in the laboratory; predictions will be made of cracking, rutting, and roughness; in-service measurements will be taken; methods will be adjusted; and new predictions and new measurements will be made until methods are calibrated or discarded.

Once we have developed a workable pavement design system, we can then determine whether many of the refinements about which we now worry are really essential or whether they are more or less window dressing for the purist. We will also be able to concentrate on what I think is an important problem—designing a positive environment for pavement that will allow us to predict more accurately the performance characteristics of the designs. With those tools, a pavement design system, and a roadway system design guide, each agency will be able to predict and finance a well-planned, well-managed service life for each pavement constructed.

Sponsorship of This Special Report

The Task Force on Structural Design of Pavement Systems is sponsor of all papers in this Special Report except the one by Freeme and Marais. That paper, sponsored by the Committee on Mechanical Properties of Bituminous Paving Mixtures, was not presented at the symposium but is included because it presents an example of the use of a fatigue subsystem.

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