SPECIAL REPORT 126

STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENT SYSTEMS

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Geologist			
Testing Engineer			
Office Manager			
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Highway Research Board of the National Research Council
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SPECIAL REPORT 126

STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENT SYSTEMS

Proceedings of a Workshop
held December 7-10, 1970,
Austin, Texas

Sponsored by the Federal Highway Administration, U.S. Department of Transportation, in cooperation with the College of Engineering, University of Texas

Subject Areas

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

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FOREWORD

For pavement designers to accommodate themselves to changing conditions, it is necessary that research in pavement design find its way quickly into the profession. These engineers are now being faced with many design problems that are outside the realms for which existing design methods were developed and that include (a) changes (increases) in loading, e. g., proposed increases in legal highway loadings and anticipated aircraft loadings; (b) increased utilization of thick layers of treated materials for more economical and longer lasting pavements; and (c) broader use of materials not currently considered, e. g., beneficiated marginal materials and new paving materials.

Moreover, in a few years many of the major highways constructed in the 1950s and early 1960s in the United States will need reinforcing, and many airport pavements will need to be overlaid and lengthened. At present no well-documented design procedure exists for thickness determination for overlays of existing pavements.

It would appear that research conducted in recent years can contribute to the development of improved methods of pavement design. These methods have the potential of providing solutions to these problems as well as providing increased reliability of the design estimate (or of performance prediction) and better definition of the role of construction including its effect on material properties.

Recognizing that within the transportation field there are a number of engineers who are concerned with pavement design and who have the necessary expertise to assist in the solution of such problems, several committees of the Highway Research Board believed that, to make these engineers more effective in bringing developments quickly to the profession and in helping to direct needed research efforts in the pavement design area, some form of interaction is most desirable at this time. An Advisory Committee on Structural Design of Asphalt Concrete Pavement Systems Workshop was appointed, therefore, to guide the program development of a workshop that would achieve the desired interaction and definition of research direction. The workshop was held at the University of Texas during December 7-10, 1970, under the auspices of the Highway Research Board and the Federal Highway Administration.

This Special Report represents the proceedings of the workshop and, in part, is designed to guide researchers into the most promising and essential areas of research and to identify for implementing agencies the wealth of information that is available today for incorporation into the design process. Part I contains a workshop summary and evaluation, a summary of the components of the design system that can now be implemented, and a summary of major research needs. The focal point of the workshop was the deliberations of nine discussion groups. Each group evaluated the state of the art in its area of concern and identified research that can be implemented or is needed. The summary reports of the deliberations of these groups are contained in Part II. The state-of-the-art papers presented to the participants on the first day of the workshop are included in Part III. Part IV presents a preliminary attempt to forecast the possible implications of the workshop on the future activities of the Federal Highway Administration and three Highway Research Board committees that were most directly involved in the development of the workshop itself. Participants and members of the sponsoring committees are given in Part V.

—C. L. Monismith

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PART I

SUMMARY AND EVALUATION

WORKSHOP SUMMARY

C. L. Monismith

This workshop has brought together a group of engineers concerned with various aspects of the structural design and performance of asphalt pavements. The remarks contained in this summary attempt to (a) draw attention to some of the highlights of the deliberations that took place during the 3½-day period in Austin, and (b) suggest ways in which these efforts might achieve one of the goals of the workshop, development of an improved framework for the structural design of asphalt pavement systems.

One of the significant aspects of the workshop is the fact that the concept of using a "systems" approach to pavement design permeated the conference. (By systems approach is meant the consideration of the complex and interrelated factors associated with pavement design within a framework permitting a systematic evaluation of distress leading to a reduction in pavement serviceability with time. Included as a part of this system are considerations with respect to safety and economic requirements necessary to ensure a given level of serviceability.) Moreover, the need to treat the pavement design-management process as one unit reflecting the influence of factors such as safety, cost, maintainability (decision criteria), and distress has been emphasized.

A question that must be answered if such a workshop is to be considered worthwhile is, "Is a mechanistic approach a necessary part of the pavement design process?" Initially it would appear that the pavement researcher had not necessarily convinced the designer that such is the case. However, from the discussions and directions of the workshop, this question would appear to be at least partially resolved in the direction of the mechanistic approach inasmuch as it appears to offer the design engineer definitive means of predicting specific modes of distress that may have a significant effect on reduction of pavement serviceability in a particular area (or environment) and to afford him the means of improving the reliability of his design estimate.

From the various presentations, it is evident that there is a large amount of information already available and that the need exists to assimilate this information into the design process. To accomplish this objective so as to make the pavement design process more viable requires, on the one hand, that the researcher work to make the information or system usable and, on the other hand, that the user of the information (the designer) have the obligation to use what has been developed. In effect, greater consideration must be given to research implementation. This will undoubtedly require much closer coordination between the designer and the researcher.

If the methods discussed at the workshop are to find their way into the design process, the implementable procedures recommended in this report must be used on a broader scale, in conjunction with existing design methods, to estimate performance. When this is done, suitable field monitoring will be required.

In the implementation process, construction and personnel costs may be a constraint. Based on experience in certain areas, however, this may not be as expensive an operation as it may appear initially.

A strong feeling was expressed in a number of the sessions that considerable effort must be devoted to define in situ conditions to ensure that a reasonable evaluation of material characteristics and pavement response be obtained. Moreover, while not discussed in detail, the idea that construction capabilities and limitations must play a role in the design process has come through "loud and clear."

The emphasis placed here on the need for relating distress to performance and performance in turn to pavement serviceability is noteworthy. This is an example of an area where the researcher and the designer must work closely together to ensure that the "right things" are measured and that correct interpretations of data are obtained.

At this point it should be noted that, although this workshop did not consider material quality requirements (e.g., asphalt mix design), such requirements cannot be separated from the pavement design process.

In a number of discussions, as noted earlier, it was stated that maintenance considerations should be an integral part of the design process. Use of dynamic programming techniques such as those mentioned during the workshop may prove fruitful for incorporating maintenance decisions into the system in a quantitative fashion.

It would appear that the interaction among the participants during the working sessions has resulted in a greater understanding on their part. It is hoped that this understanding and brief interaction will be expanded to a continuing working relationship to ensure further progress in improving the pavement design-management process.

Finally, to ensure that the most effective use is made of the resources devoted to research in this area requires that effort be directed to developing procedures whereby the research can be incorporated into the overall design framework. Accordingly, it will be necessary for groups at both the national and regional levels to develop (or expand) procedures whereby continual evaluation of what is being done can be accomplished.

IMPLEMENTATION OF RESEARCH

Advisory Committee

Each of the discussion groups was requested to evaluate the state of knowledge in its area of deliberation and to select the results of research that could be implemented to improve the pavement design process.

It should be noted that implementation of results in the context considered here refers primarily to those research results that emphasize the mechanistic approach for prediction of specific modes of distress as a part of the pavement design process and those that are sufficiently well along to be used, for example, by materials and research staff of a highway department on at least a trial basis. Moreover, these results may require verification and documentation.

The basis for the following brief statements will be found in the group reports in Part II.

GROUP A—MATERIALS CHARACTERIZATION

Many test methods exist for evaluating the stiffness and strength of pavement materials. Of these, the triaxial test is considered to represent at the present time a reasonable way to evaluate the linear elastic or linear viscoelastic parameters. Some advantages of the triaxial test follow: (a) It has been used extensively, (b) it is multiaxial in nature, (c) the principal stress ratios can be varied somewhat, and (d) it is adaptable to all pavement materials and to various means of load application (e.g., creep, cyclic, and repeated load).

GROUP B—SOLUTIONS TO BOUNDARY VALUE PROBLEMS

The present state of knowledge and technology is such that almost any relevant boundary value problem in linear elasticity or viscoelasticity can be solved. However, documentation and maintenance of available programs are usually poor. Linear solutions to the layered system problem exist in the following usable forms:

1. BISTRO—a multilayered elastic theory program developed by the Shell Oil Company;
2. Chevron 5L—a 5-layered elastic theory program developed by the Chevron Research Company; and
3. FHWA III—a 3-layered viscoelastic theory program developed at M.I.T. under a Federal Highway Administration contract.

GROUP C—DESIGN CONSIDERATIONS

Development of a working model of the pavement design system is now feasible. Examples of such models have been developed by a working group in Texas (see Part III) and as a part of NCHRP Project 1-10. Whereas the systems developed in conjunction with both projects use relatively simple formulations for the pavement thickness selection procedure (e.g., the AASHO Interim Guide), it is possible to formulate the response of the pavement structure to load and certain environmental effects in terms of available theories for mechanical behavior and distress determination.

Accordingly, immediate steps should be initiated to synthesize available knowledge into a working model of the pavement design system by using a simplified model for thickness selection that incorporates these theories.

GROUP D—LOAD AND ENVIRONMENTAL VARIABLES

The load equivalency factors developed from the AASHO Road Test equations are currently usable. There are several methods or programs available for predicting temperature distribution in the pavement system. Methods for the prediction of soil moisture beneath the pavement have been established.

GROUP E—TRAFFIC-INDUCED FRACTURE

Prediction of the number of applications of mixed traffic loadings that produce fatigue cracking with stated probabilities is now possible for asphalt-bound materials and soil-cement bases. Required input includes, among other items, fatigue tests on beam specimens, materials characterization in terms of secant moduli and Poisson's ratios, and an appropriate prediction of critical stresses and strains. However, verification of such a methodology has yet to be made on other than a limited basis, and this subsystem should only be used to provide an independent check on current design procedures.

Abnormally large vertical loads may induce shear or tensile cracking in one application. Bearing capacity analyses, such as that of McLeod, or punching shear analyses, such as that of Meyerhoff, offer an available approach to a treatment of the shear problem.

GROUP F—OTHER FRACTURE

Methods are currently available that can be used as a working subsystem to eliminate or predict the occurrence of thermal contraction cracking in new asphalt concrete pavements. One of the procedures, using a criterion for a limiting stiffness, is based on extensive field and laboratory investigations in Canada and verification in several Canadian road tests.

GROUP G—TRAFFIC-INDUCED PERMANENT DEFORMATION

Operational models are available to predict permanent deformation based on linear viscoelastic theory. Another promising method has been proposed by Huekelom and Klomp for predicting pavement rutting based on strain calculations.

Other methods are available to designers to preclude or control permanent deformations by limiting vertical strain at the subgrade or by more empirical procedures.

GROUP H—OTHER PERMANENT DEFORMATION

Solutions to the problem of identification and control of potentially expansive soils are currently available and are being used to a limited extent.

Methods for the solution of frost-heave problems based on classical approaches or currently available materials or both are readily available. Applications of present knowledge may be enhanced by a cost-benefit approach.

GROUP I—RELATING DISTRESS TO PAVEMENT PERFORMANCE

The most satisfactory method currently available for use in evaluating pavement performance is the present serviceability rating or present serviceability index.

SUMMARY OF RESEARCH NEEDS

Advisory Committee

One of the major objectives of the workshop attendees was to formulate a set of research needs that would permit the development of a methodology for the solution of a series of the most essential pavement design problems currently not solvable and for which solutions are necessary to permit the examination of specific modes of distress or combinations thereof.

The following list contains 10 major research items selected by the Advisory Committee from the deliberations of the nine discussion groups and endorsed by the consensus of the workshop attendees. Although these items were considered to be major areas requiring additional research, it should be noted that there are other research needs included in the group reports.

Ranking by order of importance was established by vote of the Advisory Committee. Although the entire group of workshop attendees agreed with the committee on the importance of these items, time limitations prevented their reaching general agreement on the actual priorities.

1. Relationship between pavement distress and a performance or failure function—The mechanistic approach to pavement analysis and design can at best yield predictions of the nature and extent of pavement distress (e.g., the extent of rutting and the nature and extent of cracking). There is an urgent need for a technique whereby such structural distress and its objective measurements (including, for example, measurements of roughness) can be related to the functional performance and perhaps to ultimate failure of the pavement. It seems apparent at this time that the only feasible way to relate distress to performance is through a statistical analysis of serviceability-performance information (most probably subjective in nature) and objective distress predictions or evaluations. Such an analysis must (a) define important distress factors involved in pavement nonserviceability and failure, (b) establish suitable weighting functions to judge the relative importance of various levels of combined distress modes, (c) identify suitable limiting levels of distress occurring separately or in combination; and (d) develop or adopt suitable measures of performance or serviceability.

2. Applicability of linear theories to predict stresses, strains, deflections, and fatigue and rutting distress in pavements—Currently well-developed and documented means for predicting the stress, strain, and deflection states within and at the surface of the pavement structure are limited to the linear theories of elasticity and viscoelasticity as applied to layered systems. Determination of the predictive accuracy of such theories by means of closely controlled and thoroughly instrumented laboratory and field experiments is a vital research need. Such a determination must include a consideration of the ability of viscoelasticity theory to predict surface rutting under both repetitive and nonrepetitive loading. Furthermore, the predicted stress or strain states or both may be used in conjunction with laboratory fatigue data to estimate the cracking of pavements subjected to repeated loading. Determination of the accuracy of these estimates is a related research need that should be investigated simultaneously.

3. Mechanical characterization of granular materials—Although it is recognized that there are many test methods that characterize granular materials, there are as yet no generally accepted methods of expressing the properties of a granular material in a manner relevant to the stress analysis problem or to the problem of cumulative deformation under repeated loadings. Sensitivity to confining pressure and various degrees of saturation, the conditioning that occurs under prior loading, and the cumulative den-

sification or distortion produced by repeated load cycles must be considered. Factors influencing material characteristics vary in time as well as in three-dimensional space. Because of the unique characteristics of granular materials in tension and compression, it is impossible to evaluate the material out of the context of the pavement system. The selection of methods of material characterization are integrally related to the mathematical model chosen to represent the pavement system and must realistically reproduce the in situ conditions of the pavement structure.

4. Effect of environment on pavement system condition and response—The environment has been evaluated in a number of ways, and temperature and moisture conditions have been estimated for the pavement section during its design period. However, few attempts have been made to evaluate the environmental factors needed for the mechanistic function of materials and pavement elements. Moisture and temperature conditions that will prevail in a pavement system under local moisture and temperature environments need to be predicted, as well as the effects of these conditions on the material properties, differential surface deformations, and pavement performance.

5. Pavement design as a stochastic process—So that designers can better evaluate the reliability of a particular design, it is necessary to develop a procedure that will predict variations in the pavement system response due to statistical variations in the input variables, such as load, environment, pavement geometry, and material properties including the effects of construction and testing variables. As part of this research it will be necessary to include a significance study to determine the relative effect on the system response of variations in the different input variables.

6. Fracture mechanisms—The mechanistic approach to fatigue crack prediction uses fracture-mechanics principles to explain the initiation, propagation, and accumulation of cracks. It offers many potential advantages as compared with the phenomenological approach, including its ability to handle both mode of loading and areal cracking and its ability to explicitly treat the stochastic nature of the process. Following the successful completion of current research programs, we anticipate additional research that will include the effects of random loading, the phenomenon of localized plastic flow due to occasional heavy loadings, accumulation of cracking for performance analysis, and continuing field verification.

7. Mechanical characterization of pavement materials (other than granular)—Although considerable progress has been made in the identification and measurement of properties of asphaltic concrete required for insertion into the linear viscoelastic and other procedures of stress analysis, there still remain important questions in the characterization of these materials and asphalt-treated base materials, cement-treated base materials, and cohesive soils. In all cases, the degree of departure of these materials from the linear response model must be determined in order to identify any deviations large enough to require special analysis. Further, the permanent deformation and fracture response of these materials to repeated loading under states of stress representative of critical states in pavement systems must be determined. The effects of the environmental variables to temperature and moisture, where appropriate, must be evaluated. After appropriate characterizations are obtained, production tests usable by highway engineers must be developed, and typical ranges of values must be determined.

8. Loading variables—Loading, as a set of input variables to the process of analysis and design, must encompass the complete history of loads anticipated during the design life including traffic volumes, lane distributions, transverse distributions within lanes, contact pressures, tire spacings, axle configurations, and weight distributions. With respect to this set of input variables, it is important to (a) determine the accuracy of existing data collection systems and the adequacy of sampling systems, (b) continue to collect relevant data and expand data collection systems as necessary, (c) refine or develop predictive methodologies, (d) determine the accuracy of the predictive methodologies, and (e) determine the required accuracy of the predictive methodologies for use in the pavement analysis and design system.

9. Methods of predicting reflection cracking—Current systems of overlay design do not provide adequate guidance in designing overlays to prevent reflection cracking. This is particularly true in the case of large random or thermal cracks found in older portland cement concrete or cement-treated base structures. In addition, current design methods do not recognize that cracking can initiate in the base course because of shrink-

age or other environmental changes. Such a crack can then reflect through the surface layer and lead to distress. It is believed that the possibility of developing a mechanistic model should be explored with the purpose of providing a rational approach to these design problems. Because portions of the Interstate and other federal and state highways are approaching the end of their structural design life, it is important that work on the problem be started at an early date so that it will be available to help in the designs that will be facing the states in the next few years.

10. Development of proper feedback information data base for the pavement system—Development of rational pavement design methods is an iterative process involving evaluation and improvement based on analysis of observed data. In the validation or modification of system and subsystem models, the large number of possible candidates for inclusion in the system require that effective selection be made of proper variables and also of compatible ways of measuring them. Effective information management here involves selection of parameters, a sampling plan (i.e., when, where, how, and why), data processing methods, input, storage, and output techniques. The pilot model of such a system in any state should involve selected pavement sections rather than the entire pavement inventory.

FEDERAL HIGHWAY ADMINISTRATION RESPONSE TO RECOMMENDATIONS OF ADVISORY COMMITTEE

The Advisory Committee on Structural Design of Asphalt Concrete Pavement Systems Workshop has submitted to the Federal Highway Administration a list of 10 major research items required for development of additional methodology for problems currently not solvable. Those items on the list, not represented as being all inclusive, are discussed in the order of priority assigned by the Advisory Committee.

FHWA EVALUATION OF RECOMMENDATIONS

The workshop panels have done an excellent job of outlining the overall needs in the program. However, recently completed and ongoing research has provided or will provide answers to some of the indicated needs. Therefore, a résumé of the position or state of the art with respect to each of the items of the list follows.

Relationship Between Pavement Distress and a Performance or Failure Function

The task of relating pavement performance or a failure function to pavement distress is a major problem area that has yet to be solved. The pavement serviceability performance concept of Carey and Irick used at the AASHO Road Test does not use a mechanistic approach to pavement evaluation. It is a correlation of quantifiable measurements on the pavement to represent the subjective opinions of the user as to whether the pavement is functioning as intended.

These serviceability-performance concepts of the AASHO Road Test do not directly take into account the pavement condition in terms of its load-carrying capability. However, attempts have been made to relate some measure of the load-carrying capability to the present serviceability index (PSI). The Road Test did some preliminary work on PSI equations with Benkelman beam deflections as one of the inputs. Several states have carried on the research in this area with some moderate success. Recent work through the HPR program with the dynaflect has shown this to be a promising tool for evaluating pavement performance.

The two most noteworthy HPR studies in this area have been the Minnesota study, Application of the AASHO Road Test Results to Flexible Pavement Design in Minnesota, and the Texas study, Application of AASHO Road Test Results to Texas Conditions.

The Structures and Applied Mechanics Division of the Office of Research has allocated funds for research, entitled Case Studies on Premature Flexible Pavement Failures, to be conducted under contract during FY 1971 and FY 1972. The basic objective of this research is to establish the causes and mechanisms associated with in-place cracking and rutting failures of flexible pavement.

This will be an in-depth study of selected pavement failures rather than a broad study of the prevalence of surface manifestations of failure on highway systems. When the mechanisms leading to specific failure manifestations are clearly understood, it will be possible to alter present design methods to alleviate these conditions in future construction.

Applicability of Linear Theories to Predict Pavement Distress

Research to determine the applicability of linear theories to predict stresses, strains, deflections, and fatigue distress in pavements has been included in studies by Monismith and Secor, Papazian, Hicks, Finn, and others. In-house research of the FHWA has been concerned with measurements of strains and deflections of pavement systems under static and dynamic loading.

Results of this FHWA research were presented at the 49th Annual Meeting of the Highway Research Board in January 1970 by Drennon and Kenis, and they indicate that the displacements are basically linear for the system tested. Considerable data have been gathered in this research, and validation of the various design theories is to be undertaken. All materials used in construction of the test sections must be characterized in terms commensurate with the available theories before complete comparisons can be made.

Fatigue theories are being investigated through administrative contracts now in progress with the Ohio State University and the University of California. It is hoped that a preliminary design procedure to ensure against failure due to fatigue will be ready for validation in the near future. This procedure can be used by design agencies to check their normal design against fatigue failure.

Inherently, theories of stress conditions cannot identify failure mechanisms associated with the stresses. Therefore, before any design method can be adopted for standard use, research must evaluate it in controlled experiments utilizing full-scale straight-line rolling loads. The FHWA is currently doing preliminary design work on a pavement test facility that will be capable of producing a given failure mechanism under controlled environmental conditions and will evaluate the ability to predict their occurrence.

Mechanical Characterization of Granular Materials

The mechanical characterization of granular materials in terms suitable for use in constitutive equations in stress analysis is considered to be a difficult task. However, the FHWA has a proposed contract to be let in the near future to develop procedures to characterize granular materials. Any test used for characterization must simulate the in-place stresses and strains of the prototype pavement. The development of the dual cyclic triaxial device, controlled by the MTS system, promises to provide an adequate method of characterization of granular and other materials. Work is also under way at the University of California to define those properties of an untreated granular material that contribute to the fatigue cracking of the asphalt concrete pavement.

Effect of Environment on Pavement System Condition and Response

The effect of environment on pavement system conditions and response is an important aspect of the overall problem of pavement design. Considerable information concerning the effects of frost and spring breakup has been obtained through the HPR program by Minnesota and other states. Also the present knowledge concerning swelling soils is probably adequate. However, it is not considered important to the basic problem of the development of a rational design method that we know the details of the environmental effects. Later, limiting environmental criteria must be developed as well as limiting properties of materials affected by the environment.

Pavement Design as a Stochastic Process

Stochastic methods must eventually be used in any rational design method. However, the development of the method need not be delayed until all statistical variations of loading, materials, and processes are known. Statistically valid associations among the various factors will be one of the refinements of the design process after it has been established. However, much information concerning the variations of thickness, density, Marshall stability, gradation, and moisture properties has already been gathered in HPR research in the quality assurance program. One of the major problems still to be attacked is that of the relationship of these variations to performance properties of the pavement.

Fracture Mechanisms

Ohio State University is currently using fracture mechanics in its study, Development of Testing Procedures and a Method to Predict Fatigue Failure of Asphalt Concrete

Pavement Systems. This research and that at the University of California, Basic Properties of Pavement Components, have resulted in a method of predicting a "time to first crack." A follow-on contract now being negotiated with Ohio State University, Analysis of Fatigue and Fracture of Bituminous Paving Mixtures, has as an objective the development of methodology for predicting cracking under repeated loading using fracture mechanics concepts. This FHWA-funded research should produce a first-generation method of predicting fatigue fracture of flexible pavements. Further effort will be needed to evaluate the method and refine it for general use.

Mechanical Characterization of Pavement Materials (Other Than Granular)

Mechanical characterization of pavement materials (other than granular) is still considered to be a major problem, although methods have been developed for characterization of these materials in both the elastic and viscoelastic states. A contract, Characterization of Asphalt Concrete and Cement-Treated Granular Base Courses, with Materials Research and Development is nearing completion. Work on materials characterization has also been undertaken at the University of California in its research project, Basic Properties of Pavement Components.

Further research in this area will be funded during FY 1972 in a study, Development of Procedures for Characterization of Untreated Granular Base Course and Asphalt Treated Base Course Materials, for which requests for proposals are now being prepared. Later it will be necessary to establish probable ranges of characteristic values for given conditions of materials, stress, and environment. Another study, Translating AASHO Road Test Findings—Basic Properties of Pavement Components, conducted by Materials Research and Development for NCHRP has also been concerned with material characterization. The ongoing research is an outgrowth of this work.

Loading Variables

The problem of identification of loading conditions or patterns has been studied for years by the states and the Office of Planning of FHWA. Loadometer studies present load data in both actual loads and 18-kip equivalents. However, the load data are samples of the total loads on any given road section taken at both permanent and mobile scale sites.

Encouragement will be given to the initiation of studies to (a) evaluate the accuracy of overload data presented in the W-4 Loadometer Tables; (b) evaluate the accuracy of current load and traffic projection methods; and (c) determine the probable error in transfer of load and traffic data from master stations to construction locations.

Methods of Predicting Reflection Cracking

A method of predicting reflection cracking is another of the recommended items not necessarily germane to the development of a rational method of pavement design. However, this will be one of the major problems facing the highway industry in the next few years and is one of the FHWA high-priority research areas. Several HPR and experimental research projects are currently under way, and more projects will be promoted. Various methods of reinforcing pavement surfacing have been tried as well as methods of breaking the bond between the old surface and the new, especially near the cracked area.

The development of a rational pavement design method will also enable better understanding of the forces at work in the reflection cracking problem and will provide insight as to the pavement properties necessary to resist them.

Development of Proper Feedback Information Data Base for the Pavement System

The development of a data base, specifically the development of a data acquisition and storage system, is another of the items that can better be regarded as a refinement problem rather than basic to the development of a design method. The concept of such an information data base for the pavement design system cannot be questioned. There are, however, difficulties to be overcome before such a data base can be initiated. Data germane to a given purpose must be derived under circumstances and conditions that are also germane.

Although the FHWA did not support the National Satellite Study Program to extend the AASHO Road Test design concepts to local conditions, support was given to a limited number of documented AASHO satellite sections constructed with more than normal job control. Much detailed information covering the performance of these documented sections throughout the country has been collected. This work and documentation can be used as a start toward the data base. Examination of the existing data can tell what we have and what it is worth. It will also point out additional data that are lacking and should be obtained. Methods to systemize and centralize the collection of existing data from these sections must be made. Minnesota, Illinois, Texas, and Iowa are currently studying the capabilities of several data acquisition systems.

FHWA RESEARCH

In 1965 the Bureau of Public Roads initiated a national program of research "to evolve a rational design procedure for flexible pavements which, on the basis of measurable properties of the materials, traffic loadings and environment, will reliably predict performance." It was considered at that time that answers to the following questions were needed:

1. What are the pavement component material characterizations under traffic-induced stresses, and how do these characterizations affect pavement system behavior?
2. Which stress distribution theory will adequately describe the behavior of the pavement system under traffic loading?
3. What is the effect of the environment on pavement system behavior, and how does it influence pavement performance?

A general plan of research for obtaining the answers to these questions was developed. It encompassed the following:

1. Special research reviews to develop insight on the current state of the art and to point out where emphasis should be placed in future research;
2. Solutions of stress distribution theories by development of usable expressions for the computation of stresses and deflections in a flexible pavement, with consideration given to theories covering linear and nonlinear elasticity, linear and nonlinear viscoelasticity, thermoelasticity, elastoplasticity, and thermoplasticity;
3. The identification of instrumentation essential to the measurement of stresses, deformations, and forces in both model and prototype pavement systems and the development of new instrumentation where necessary;
4. The development of special laboratory tests to characterize the behavior of pavement component materials under conditions of stress and environment that exist in pavement systems in service;
5. The study of environmental conditions and their effects;
6. The study of the effect of pavement profiles and vehicle suspensions on the magnitude of the dynamic loading and the response of the pavement system to these dynamic loadings;
7. The comparison of stress and deformations predicted by theories with those measured in pavement systems; and
8. The development of a design theory based on insight into the problem provided by the results of these studies.

A study of the plan of research, the research needs items recommended by the Advisory Committee, and the research accomplished by state highway departments, contractors, and FHWA indicate that the original objective of the program is still valid and that the research plan is still valid though in need of updating and refinement.

COMMENTS

John M. Griffith and Alfred W. Maner

The proceedings of the workshop represent a good report on the present status of asphalt pavement structural design research. The original purpose for holding the workshop was to encourage cooperation and coordination among the various Highway Research Board committees responsible in this area. The workshop did a great deal to further this cooperation and coordination. At the 50th Annual Meeting of the Highway Research Board, there was considerable effort in this direction among all the committees involved and a few committees not directly involved. A second purpose, that of providing the Federal Highway Administration with guidelines for its research program, was also adequately served.

The big question to be considered is: Who will assume responsibility for seeing that something useful is done with the findings of the workshop? The Federal Highway Administration response indicates the possibility that FHWA might do this. It seems to us that FHWA is the only organization in the United States that has the necessary resources to do so. The Asphalt Institute has done its best to assume this leadership role and, within the limitations of its resources, has made considerable progress. This program, worked up in 1969, is quite consistent with the outcome of the workshop. We would hope, however, that the much greater resources of FHWA would be directed to the objective in as vigorous and positive a manner as possible. In our opinion it is entirely within our present capability to develop a working theoretical structural design procedure for asphalt pavements within a reasonable time period, i. e., no longer than 5 years. We concur in this regard with the following comments by James F. Shook in his report of Group C.

> The group chairman recommends that immediate steps be initiated by FHWA to synthesize available knowledge into a working model of the pavement design system by using as much as possible of the knowledge discussed at this workshop, that this system be made available to designers and researchers, and that study and use of the system be encouraged to determine whether we are on the right track. This step might show that many elements in the system now considered to be important may not be so important and many considered to be unimportant may be more important. Some steps thought impossible may prove to be possible, and vice versa. We should stop looking at pieces of the puzzle, start putting the puzzle together, and see what the picture looks like.

Clyde N. Laughter

My first comment is that this workshop exposes how little we know about something we have been doing for years. Even in some of the more scholarly presentations given here, we read of "theories" that careful inspection reveals to be perhaps "hypotheses." Some patterns of research and observation have been put in focus if not in agreement.

Second, for the working engineer the importance of variables has been presented, although the ranking of importance is still somewhat murky. The same crystallization unfortunately is less clear for many fundamental definitions that still haunt the man in practice. These include distress, serviceability, failure, performance, adequacy, elastic, stochastic, micro, and macro.

Third, I feel workshops like this whet observation and analysis and should change my approach and make me more observant of new developments. Again, at the working level, I will never use Monte Carlo, Russian roulette, and the like, but I am aware that researchers with proper tools will do so.

One almost feels deflated, perhaps defeated, when the number and interaction of variables are noted. Moavenzadeh's presentation is an example of this. It should serve as a caution, perhaps a brake, to those naive souls who, in well-intentioned but poorly planned and inadequately financed research projects, unsheath rather wooden swords of three or four variables to destroy our rather complex dragon. With modern, automatic high-speed data accumulation, sorting, and evaluation, there is no reason that several hundred variables cannot be studied in pavement evaluation. (Admittedly this requires vast consolidation of variables.)

Workshops of this kind are most necessary to stimulate thought and point up directions to take and should be implemented again.

Harry A. Smith

The primary stated purpose of the workshop, "... to improve the interaction between groups of engineers and researchers concerned with the design of asphalt concrete pavements," was accomplished. In my opinion, the results of the interaction will be (a) early implementation by highway departments of the pavement management systems concept by using current empirical relationships as the structural subsystem input, and (b) better management of research programs leading to a more rational approach to the structural subsystem mainly because of improved awareness by the academic community of the real-world pavement design process. It is in this context that I offer the following comments on the Group B sessions.

It is unfortunate that the relationship of the solution of boundary value problems to the overall pavement design process was not considered an appropriate item for discussion early in the session. There seemed to be a feeling that the solution of a boundary value problem and the design of an asphalt concrete pavement are synonymous. It is my understanding that the boundary value problem solution can be used to predict behavior of a pavement structure in terms of stresses and strains when subjected to known loads and ultimately to predict the initiation of distress as exemplified by cracking and distortion. The designer must predict the performance of a pavement that is to be subjected to the complex interaction of load, environment, and time variables. Until there is a suitable procedure for relating predicted pavement behavior and distress to pavement performance in the real world, I cannot see how the solution to boundary value problems will be useful to highway pavement designers. For this reason, the state of the knowledge in this field can be described as follows:

1. From an academic standpoint, solutions to the linear elastic layered system problem for stress boundary conditions and for uncoupled thermal stresses exist in a variety of forms. From a pavement design standpoint, they are of little practical value at present because (a) their ability to predict pavement behavior and distress under real-world load, environment, and time conditions has not been adequately verified; (b) relationships between pavement behavior and pavement performance have not been established; and (c) the necessary material characterizations have not been accomplished on a wide scale.

2. Solutions to more complex boundary value problems that more closely approximate the pavement structure are available in only a limited sense and require extensive computer capability.

3. The actual use of solutions to boundary value problems in the design of asphalt concrete pavements is currently limited to special problems for which there is very little or no performance experience to influence the design. Other highway pavement design procedures that appear to be based on linear elastic theory are in fact so simplified by a variety of assumptions and so dependent on correlations with performance

of pavements subjected to similar loads and environmental conditions and built with similar materials that the procedure can be used only for the experience gained. Such a procedure does not appear to have any benefits over current empirical procedures.

In view of the need for relating predicted pavement behavior and distress to pavement performance before boundary value problem solutions are useful to the designer, it becomes extremely difficult to visualize short-term benefits from application of simplified linear elastic theory. However, elastic and viscoelastic theory could provide a means for developing methods to modify empirical design procedures to partially account for time- and climate-dependent variables. This would result in improvement of the structural subsystem in the short-to-intermediate time period while researchers are developing and verifying a more rational method for the subsystem and analyzing the sensitivity of the overall system to the structural subsystem.

The following statement is included in the address by Pister: "Thus, it appears that this workshop was based on the implicit assumption that a rational method for pavement design exists, is important to acquire, and is accessible to the minds of engineers." It is in this regard that boundary value problem solutions offer potential for significant improvement of the structural subsystem of the pavement design process. However, in my opinion, researchers should proceed to more complex solutions that have the potential for long-term payoff.

Current research programs should be evaluated in terms of relevance to the pavement design process and potential for usefulness in both the short-term and long-term periods.

Future research also should be planned and programmed to be relevant to the real-world pavement design process. Research in the area of solutions to boundary value problems should be conducted in the short term as follows:

1. Develop methods for modifying currently used empirical design procedures for the structural subsystem to accommodate time- and climate-dependent variables; and
2. Develop simplified methods for limited modification of empirical relationships used in current design procedures.

In the long-term approach, research should be conducted as follows:

1. Develop procedures for converting predicted pavement mechanistic behavior and distress to pavement performance; and
2. Initiate efforts aimed at determining the feasibility of developing a more rational approach to design of the pavement structural subsystem that would (a) provide a sound basis for extrapolation of useful data on loads, time, and environment, (b) permit the use of new materials and those having modified properties with confidence, (c) consider to a greater extent inputs such as load dynamics, and (d) permit more precise design of the structural subsystem and reduction of pavement management cost.

F. P. Nichols, Jr.

I appreciate the opportunity to offer my comments on this workshop. It is encouraging that such a conference has been held and that so many viewpoints have been brought together.

W. N. Carey made a strong point in his opening remarks to the workshop participants. He emphasized the need for satellite road tests to evaluate materials behavior in the "real world." In view of the report from Group A, particularly regarding the need for verification of new theory through full-scale field test sections, it appears that satellite studies are no less necessary now than in 1961 when the AASHO Road Test was finished and the AASHO Interim Guide (FHWA's bible) was issued.

In most theoretical approaches, there is a tendency to characterize materials for use in flexible pavement structures almost wholly in terms of their ability to minimize deflection at the surface. An unbound granular base material, which should never fail by fracturing or shearing if adequately surfaced, might be rejected solely because the wearing surface is too inflexible to stand the deflection. If we are to continue use of the term "flexible pavements" and if we are to use to the best advantage bases that distribute loads through aggregate interlock and particle friction, I suggest that research to improve the long-term flexibility of the bituminous layers should be assigned high priority.

By the same resistance-to-deflection criteria, bases composed of nonuniform, marginal quality aggregates bound together with asphalt or cement might be chosen in preference to unbound bases of quality controlled aggregates, even though the latter are still available at far less cost. This might result in reductions in the apparent total thickness requirements but simultaneously, in certain cases, in more costly materials being "dumped in the mud." Obviously the performance of thin, semirigid bases over highly variable subgrade soils (as described by Sherman) will be far from uniform.

The dilemma facing the theoretician trying to "sell" new, more rational pavement design methods is well expressed in the Group B report, which points out that highly mathematical theories tend to scare off the design engineer whereas oversimplification of these theories results in loss of credibility. The report of Group I recognizes that new methods will not "sell" unless they show benefits over the old.

My personal conviction, in view of the variables and of the many other imponderables involved, is that empirical methods, perhaps based on CBR or similar, more realistic strength test criteria, are not so irrational after all and should not be abandoned in favor of unverified theoretical approaches.

Frank H. Scrivner

The listing of research needs is certainly relevant and the individual items are directly related to the overall problem. However, item 10, which we in Texas call the "feedback data system," should have a higher priority. Each state should have a central storage place, which is easily accessible to the engineers and the researchers operating in that state, for traffic, performance, maintenance, and cost data collected systematically on selected sections of the state highway system. Such a data system could be useful in the design and management of pavements long before the other items in the priority listing have been completed.

George B. Sherman

I note that, in the Federal Highway Administration's critique of this workshop, the overlay for reflection cracking was not considered a part of the structural design problem. I tend to disagree with this because the reflection crack mechanism is one that can possibly be applied to the current design methods for determining the thickness of asphalt concrete over cement-treated base. It is also probable that any theories developed for flexible pavement design will apply to the overlay problem.

The question was raised concerning why the need for a structural overlay system for flexible pavements was not identified as a primary research need. In this area a number of agencies appear to be applying rational methods for devising the thickness of overlay to correct structural deficiencies. These are the RTAC, The Asphalt Institute, and the state highway departments of Oklahoma, Utah, and California. All of these systems require a basic measurement of deflection to determine the overlay criteria.

A more comprehensive pavement overlay design system was developed by B. F. McCullough in his doctoral thesis. This system considers wheel loads, temperature changes, and performance. The structural overlay design problem, therefore, seems to be well covered, whereas the reflection cracking problem, which requires analysis of a different type of failure mechanism, is not. This, I believe, was the reason for the emphasis given to this problem by at least two of the groups at the workshop.

PART II

DISCUSSION GROUP REPORTS

GROUP A
MATERIALS CHARACTERIZATION

Chairman, Richard D. Barksdale; recorder, E. J. Barenberg; members, J. A. Deacon, Fred N. Finn, J. E. Fitzgerald, C. R. Hanes, Richard A. McComb, Wolfgang G. Knauss, Roger LeClerc, Kamran Majidzadeh, Fred Moavenzadeh, Karl S. Pister, James M. Rice, and Ronald L. Terrel

Materials characterization has long been an integral part of the pavement design process. Many of the tests currently used to characterize paving materials are empirical in nature and, as such, do not measure fundamental properties of these materials. This is due in large part to the lack of development of a pavement design procedure based on a mechanistic model of the pavement system. As a consequence, arbitrary test procedures have been developed to evaluate materials for use in empirical procedures.

The current interest in developing a rational approach to pavement design has created a need for using more fundamental approaches to characterize paving materials. The selection of methods of material characterization is integrally related to the mathematical model chosen to represent the pavement system. Therefore, the materials engineer should consider the mathematical model that will be used to predict the pavement response when he selects appropriate test methods.

Characterization of paving materials is a complex problem. The rate dependency, environment, effect of stress state, and effect of time on the material properties all add to the complexity of the problem. Characterizing a material under laboratory conditions is relatively easy compared with realistically characterizing the true material response in a pavement system over a long period of time. Stresses, strains, temperature, rates of strain, levels of stress, and other critical factors that influence material characteristics vary not only in the three-dimensional space domain but also in the time domain. Thus, the testing method and conditions under which the test is performed in the laboratory must be chosen with great care to represent realistic conditions of these materials in service.

A unique characterization of paving materials is not possible. All material characterizations used in representing paving materials must by necessity be a compromise between rigor and practicality. Usually the compromise is based on average or limiting conditions that are believed to be critical with respect to pavement behavior. Sensitivity analyses should be performed and the results evaluated so that they can be used to guide the engineer in selecting the most representative conditions. This implies that the mathematical model chosen to represent the pavement structure is capable of doing so with reasonable accuracy.

The development of mathematical models requires certain assumptions with regard to the behavior of the material. If the behavior of the material is not in agreement with the assumptions made, revisions will have to be made of the governing model. In many instances it may be necessary to approach the problem on an iterative basis. That is, it will be necessary to evaluate a pavement system with a simplified theory as a first-order approximation to determine the general states of stress and strain in the system. These results can then be used to guide the investigator or engineer in selecting the necessary conditions for further characterization of the materials. More sophisticated pavement analyses can be made with the refined material properties and new conditions for material evaluation until a satisfactory degree of accuracy is obtained. This method has been applied reasonably accurately by using elastic theory to evaluate pavement systems composed of viscoelastic materials.

For practical design procedures, tests for characterizing material properties must be suitable for testing on a production basis by highway department personnel. Testing procedures must be such that testing errors do not camouflage the true properties of the materials. Equipment for production testing must be inexpensive so it can be purchased or developed by highway departments and other agencies charged with pavement design.

The members of Group A were drawn from a broad background of experience and disciplines ranging from material scientists from the aerospace industry, to specialists in the field of mathematical modeling techniques, to practicing highway engineers. This broad spectrum of backgrounds resulted in a lively exchange of ideas and a blending of viewpoints.

CURRENT STATE OF KNOWLEDGE

The current state of knowledge with regard to material characterization is nicely summarized in the paper by John A. Deacon. As he points out, the response of paving materials is influenced by rate of loading, temperature, environment, and stress state. Because a three-dimensional state of stress exists in the material beneath a pavement subjected to a moving load, it was agreed that the test procedure used should be multiaxial in nature, with provisions made for changing the ratio of principal stresses in the test. The triaxial cell was considered to be a reasonable testing device to use in characterizing material properties, at least for the present time.

Based on current knowledge and a consideration of the current capabilities for solving layered systems problems, a promising method for material characterization appears to be the use of linear viscoelastic parameters. By testing these materials with a creep, a cyclic load, or some other type of test, we can evaluate the necessary parameters for characterizing the material for use in a linear viscoelastic layered system model such as the one developed by Moavenzadeh and now under verification by the Federal Highway Administration.

Discussions in the work sessions indicated that the response of pavement systems could be estimated to a reasonable engineering degree of accuracy by using an iterative procedure in which successive approximations of the secant modulus are an input parameter for the elastic layered system model. The secant modulus used in this method is measured with the cyclic load triaxial test. This approximate method has already been applied to pavement design problems by a number of investigators. It was also brought out in these discussions that rutting can be estimated by using a linear viscoelastic layered system model, as were the appropriate parameters to describe the linear viscoelastic properties. Furthermore, with further extensions of the theory, fatigue cracking may also be predicted by using viscoelastic theory.

Characterization methods for granular (unbound) materials are in a poor state of development at the present time. It was agreed that, because of the unique characteristics of granular materials in tension and in compression, the behavior of the material must be considered in the actual pavement system. After considerable discussion, it was agreed that the probable states of stress for the material must be determined before the test procedures can be established.

Empirical tests as currently applied to paving materials are probably of little value in developing rational design methods unless they measure properties that can be used with elastic or viscoelastic layered system models. The important contribution in the future of empirical tests will probably be in the quality control area of materials engineering. Some current test procedures, such as the California resilience test, may have value for characterizing material properties for use in rational design methods. Triaxial tests performed by using repeated or creep loadings also appear to have much potential for evaluating some pavement materials.

RESEARCH NEEDS

Fatigue fracture of stabilized materials used in flexible pavements on a nationwide basis at the present time appears to be a more important cause of structural deterioration than is the accumulation of permanent, load-related deformations. This is not to imply that rutting of the pavement structure is not an important consideration. In certain areas rutting is probably as important as fatigue or even more important. Furthermore, rutting may become a more important consideration as we go to deeper asphalt concrete sections and as current practices are modified to reduce fatigue fracture.

Fatigue fracture was considered by Group A to require immediate and also long-term research studies. Considering the results of previous research in this area, we thought that emphasis should be placed on formation and propagation of cracks and overall characterization techniques that give results compatible with field observations. Because of time limitations, Group A did not delve into this subject in detail inasmuch as Group E had already considered this aspect of pavement design.

Characterization of rutting is a very complex problem. If the problem is solved reasonably rigorously, a linear viscoelastic or even a more complicated model must be used. An alternate approach to estimate rutting would be the use of an engineering method similar, for example, to that currently used to estimate fatigue life. In any case, much work will be required to develop and verify a suitable technique. The use of a linear viscoelasticity characterization probably offers the most straightforward method at the present time. Currently, a creep test is being used by some to evaluate the linear viscoelastic response. The application of this method to characterization of all materials, especially granular ones, remains to be verified for large numbers of load applications and field conditions.

Before any material characterization and accompanying theory can be accepted in a design procedure, careful verification of the theory will be required. Verification of the theory, including material characterization, can be done by the use of both accelerated, environmentally controlled, full-scale model studies and full-scale field tests. Serious doubts, however, were raised concerning whether a design procedure can be verified by the use of model studies. Therefore, the construction, instrumentation, and monitoring of full-scale field test sections are necessities and require immediate attention.

Another problem requiring immediate work is that of characterizing granular materials. A detailed study is needed of the applicability of currently used methods for characterizing the response of these materials when subjected to various states of stress. Based on this study, realistic methods should be selected or developed for characterizing granular materials. This problem, however, was not considered to be as pressing as those previously discussed. Work in the immediate future was considered to be necessary. Because of the complicated nature of the problem, a continuing program of research will probably be required over a relatively long period of time.

Finally, a closer examination of Poisson's ratio was considered desirable as a long-term research objective. Most present studies have indicated that the pavement response is not extremely sensitive to reasonable variations in this parameter. When characterization tests are carried out, however, every readily available opportunity should be taken to study this material property.

SUMMARY AND CONCLUSIONS

At the present time, material properties can be realistically characterized to a reasonable degree of accuracy for use in the more commonly applied layered system theories by means of a cyclic load test. Only a very few attempts have been made to use this or other approaches to characterize rigorously either granular materials or cumulative permanent deformations caused by the repeated application of wheel loadings. Considerable progress has been made in characterizing fatigue behavior of stabilized materials.

Fatigue fracture has been studied by applying repeated loads until failure occurs in both supported and unsupported beam specimens and also circular slabs resting on varying types of simulated subgrade supports. The results of these tests, however, indicate that a number of important questions still remain to be answered concerning fatigue behavior.

From the discussions of Group A the following research areas were determined to require immediate study:

1. Fatigue fracture of stabilized materials,

2. Verification of material characterization and design methods by full-scale model and field studies, and
3. Cumulative permanent deformations.

The characterization of unstabilized, granular materials was also considered to require a concentrated research effort, although the solution to this problem was not considered to be as critical as those areas listed.

As more knowledge of pavement behavior and response is gained, characterization procedures should be periodically reexamined and, if necessary, revised to yield a model that more nearly describes the actual behavior of the material. As more information from model and field studies is accumulated, the necessity may arise for the development of new tests and characterization procedures. For these reasons, the development of material characterization techniques and also the development of the pavement design method in general should both be considered as a continuing process.

GROUP B

SOLUTIONS TO BOUNDARY VALUE PROBLEMS

Chairman, Robert L. Schiffman; recorder, William H. Perloff; members John E. Burke, Milton E. Harr, Phillip L. Melville, Keshavan Nair, Frank H. Scrivner, Harry A. Smith, N. K. Vaswani, Harvey E. Wahls, and Russell A. Westman

A proper theory for use in the structural design of asphalt concrete pavement systems would provide numerical results that can be used by the design engineer to produce a design or a design procedure. Specifically, the process of development of a design procedure requires a sequence of steps. First, a theory of behavior must be promulgated. This requires that the materials involved be appropriately characterized. Once this is accomplished, the material characterization is applied to the appropriate principles of mechanics and probability theory to form a set of equations that constitute the governing mathematical relationships for the predicted behavior of the pavement system. The mathematical equations will be capable of predicting behavior only within the limits to which the components of the equations accurately represent real-world behavior.

The boundary value problem for the structural design of asphalt pavement systems is composed of the governing equations described and the boundary and initial conditions expected to occur in a real system. A properly posed boundary value problem requires both a proper set of governing equations and a proper set of boundary and initial conditions.

Once the boundary value problem is established, it must be solved to produce numerical results. Ideally the solution produced by using analytical or numerical techniques provides a means of predicting pavement performance under a wide variety of expected circumstances and conditions.

Theoretical approaches to pavement design problems have the following two functions: (a) the application of known principles of continuum mechanics and probability theory to the development of specific boundary value problems, and (b) the development of new analytically based theories of pavement response that can lead to new methods of pavement design.

In the first instance the theory is used to predict the response of a given pavement or pavement type to a given environment. To fulfill this function, one must be able to call on the proper theory, translate the theory into numerical terms, and develop a quantitative prediction for a specific engineering situation. In addition, the validity of the theory being used must be assessed. In this way limits of uncertainty can be placed on the behavioral predictions. For example, a pavement designed to carry light traffic over firm foundations, such as rock, might reasonably be analyzed within the framework of the linear theory of elasticity. On the other hand, an asphalt pavement constructed in a climate subject to temperature extremes might be analyzed by giving consideration to thermal viscoelastic theories and large deformations. Because such theories are not as yet tractable for predictive purposes, simpler linear theories must be used. It is then necessary to place bounds on the validity of the approximation.

New theories should meet three interdependent objectives:

1. The new theory must have a higher degree of predictability than the competing existing theory;
2. The new theory must be viable; and
3. The new theory must be credible.

The first objective is to develop a theory capable of predicting behavior of real materials. In reality, this means the extension of existing theories to reduce the restrictiveness of the physical assumptions of the existing theories. For example, viscoelastic and elastoplastic theories have a greater range of predictability than do elastic theories.

In addition, the new theory, as formulated in a boundary value problem, should realistically formulate the real world. This can be done deterministically by establishing bounds on response. A more realistic formulation would apply probabilistic principles to natural occurences.

To be viable, a theory must be capable of producing numerical results. This requires that consideration be given to the techniques by which numerical information can be culled from a theory, new or existing.

If a theory is to be applied to design situations, it must be credible to the design engineer. Credibility is often a subjective entity. A highly mathematical theory presented in an obtuse manner will probably frighten the design engineer. On the other hand, an oversimplification of the physical processes involved causes the theory to be less credible to engineers who are empirically knowledgeable of the physical factors involved in the action of pavement systems.

The development of techniques of solution is not confined to any particular method or level of development. It may be analytical in the sense of developing general methods of solution to a large class of boundary value problems. On the other hand, this development may be concerned with the solution to a particular boundary value problem. Furthermore, there is no restriction as to the means of solution. They may be numerical, analytical, or a combination of both methods.

In summary, the role of theory as used here is largely mathematical. Its range extends from the formulation of new theories to the implementation of existing theories. The theories used are derived from continuum mechanics and probability theory. Methods of solution are those of pure and applied mathematics, numerical analysis, and computer science; and the tools are those of calculation including the use of computers and computational systems. The output is numerical data.

CURRENT STATE OF KNOWLEDGE

The current state of knowledge of solutions to boundary value problems that have applicability to asphalt concrete pavement systems is largely the state of knowledge that exists in solid mechanics as tempered by the ability to provide numerical output to a given relevant boundary value problem. Just as solid mechanics provides a quantification of descriptive physical relationships, a computational algorithm provides numerical results for symbolic algebraic representations.

Various surveys of the literature exist (1, 2, 3). These surveys are in a continual state of updating by the Highway Research Information Service (HRIS).

Types of Problems Solved

Boundary value problems with applicability to asphalt concrete pavement systems are concerned with the response of a layered continuum to a set of defined boundary forces or displacements and environmental fluctuations or both of these. Within the context of continuum mechanics, the problems of concern are conveniently classified in terms of the constitutive laws that are adopted, as follows: linear elastic, linear viscoelastic, plasticity, and nonlinear.

Solutions to the linear elastic layered system problem for stress boundary conditions exist in a variety of forms. These solutions have provided numerical information on a variety of parametric effects including surface loading conditions, layer interface conditions, and material homogeneity. In addition, solution techniques and a limited amount of numerical data exist on the effects of displacement boundary conditions, thermal environments, and poroelasticity. The thermoelastic problems are all based on theories that separate the effects of load and temperature as independent activities. Thus, they use an "uncoupled" theory. No experimental or other evidence exists that bears on the question of whether the thermoelastic boundary value problem, as applied to asphalt concrete pavements, can realistically be uncoupled.

Solutions to linear viscoelastic layered system problems for stress boundary conditions also exist. Because of their comparatively recent origin and their higher level of complexity, the viscoelastic case has not been studied as extensively as the elastic case. Viscoelastic solutions have been largely used to assess the role of vehicle motion and cumulative, time-dependent deformations.

Some solutions to plasticity problems exist. These use deformation relationships and have been developed for finite element and finite difference techniques.

Some anelastic boundary value problems using incremental relationships have been solved. In general, however, the solutions are for isolated, specialized problems.

Status of Computer Programs

There are essentially no boundary value problems that apply to asphalt concrete pavement systems that do not require a procedure for a moderate-sized computer. (Machines of "moderate" size are the Burroughs B5500, the CDC 6200, the IBM 360/50, and the Univac 1106.)

The status of a useful computer program is a direct function of the availability and usability of that program in a state highway department environment. This requires consideration of the following aspects of program status:

1. Machine compatibility—the degree to which a given program is dependent on a given machine installation or machine vendor;
2. Program documentation—the written material that must accompany a program to make it understandable and usable; and
3. Program maintenance—provision of a mechanism for removing programming errors (bugs) and provision for an orderly transition of program operation as computing installations upgrade hardware and change their operating software.

It is an unfortunate fact that the state of computer programs for solving layered system problems is deplorable. There are several public domain programs for the solution of layered elastic systems. One of these programs is multivendor compatible. The state of documentation and maintenance on all of these programs is poor to nonexistent.

There is at least one proprietary program for solving layered elastic systems. Negotiation for the purchase or lease of these types of programs can include machine compatibility, documentation, and maintenance.

A single source program for the solution of layered linear viscoelastic systems exists. This program will develop numerical values for stress boundary conditions under quasi-static conditions including moving loads. This program has been implemented on a single-machine configuration. It is documented, but it is not maintained.

Applications

Layered systems analysis, as part of a design procedure, is being used by two states, Kansas and Kentucky, the U.S. Navy, and the Shell Oil Company. In all cases, an idealized version of linear elastic systems boundary value problem solutions is used. The idealizations concern the number of layers in the system and the material properties. At most, three layers are considered.

POTENTIAL FOR MODIFICATION OF CURRENT DESIGN METHODS

Of the boundary value problem solutions available or potentially available, only linear elastic and viscoelastic solutions have a potential for implementation in design procedure in the near future.

Elastic Solutions

There is a potential for the use of elastic theory to modify current design methods. The important variables that could be included in a design modification are as follows:

1. Material properties—elastic constants and thermal properties and the ratios of these properties in adjacent layers;

2. Interface conditions—consideration of interface separation and slip;
3. Surface conditions—consideration of adhesion and friction between the tire and the pavement and the effects of nonsmooth surfaces; and
4. Displacement—consideration of the relationship of mixed to stress boundary conditions.

Linear elastic theory is a wholly self-consistent theory. Thus, within its assumptions, it contains a basis for performing parametric studies and developing the stress distributions that are useful for failure studies. Furthermore, the self-consistency of the theory permits sensitivity analyses to be performed, which can be used as a basis for extrapolation.

Viscoelastic Solutions

In addition to having the attributes of elastic theory, viscoelastic theory also provides the capability of predicting accumulated deformations for loads moving at slow speeds and for temperature changes. The important additional variables to be considered are the material operators of both the pavement and the tire and the energy transfer between the tire and the pavement. Early time effects are of particular interest.

POTENTIAL FOR THE DEVELOPMENT OF NEW DESIGN METHODS

Two areas of investigation show potential for future developments of new design methods: fracture analysis and the use of stochastic techniques.

Fracture Analysis

Fracture analysis would provide estimates of crack growth, reduce the number of tests to evaluate fatigue, and assist in maintenance planning. The variables of importance are the character of the flaw, the material characteristic, and the load and environmental history.

There are two phases to a fracture analysis: the application of an uncracked stress field in a field of cracks and the consideration of the influence of cracks in a total layered system.

Stochastic Techniques

The objective of a stochastic procedure is to develop a set of output statistics that are the consequence of a set of input statistics. Hardly any of the entities involved in the analysis and design of pavement systems are deterministic. All of these entities are subject to random variations of one form or another.

A realistic view of the constitutive relationships must assign a degree of random variation to the numerical values for the material properties and the form of the constitutive relationships. This randomness may be due to quality control and is thus a property of the specific material. It also has an areal distribution connected with geologic variations, the methods of construction, and the location of aggregate supplies.

The boundary conditions are also subject to randomness. Traffic patterns are not deterministic. In addition, environmental conditions such as climactic and water conditions can only be established in a proper manner by considering their random nature.

A stochastic technique would include such probabalistic entities in the appropriate boundary value problem. The solution that is developed will provide stress, strain, and displacement as statistical entities. That is, each numerical value of pavement response would carry a probability of occurrence. Thus, a theory of pavement behavior would assign a level of confidence (or probability) to a particular predicted occurrence. This carries over to design in which the confidence of behavior is applied to a confidence in the design.

RESEARCH DIRECTIONS AND NEEDS

To develop research needs requires an assessment of current directions of research.

Current Research Directions

The directions of current research are as follows:

1. Development of solutions and solution techniques for boundary value problems that use increasingly complex constitutive equations (the constitutive equations being used are more realistic than their predecessors);
2. Development of solutions and solution techniques for boundary value problems that use increasingly complex geometric configurations;
3. Development of computational techniques involving new methods for solving old problems and newly formulated problems and techniques for information handling (data structuring) by computer to provide more efficient machine codes;
4. Development of comprehensive user-oriented man-machine computer systems including interactive computer graphics techniques;
5. Development of global methods that examine the response of the pavement system as a total entity; and
6. Evaluation of the predictive capability of existing theories.

Areas of Needed Research

The areas of needed research are divided into long- and short-range topics. No attempt is made to define the time required to develop these tasks. This is a question of indeterminate variability. Research with short-term goals includes the following:

1. Extension of currently available mechanistic methods to be within the reach of practicing highway engineers by determining the predictive capability of currently available mechanistic methods and by documenting and maintaining computer programs that are machine-independent;
2. Evaluation of the payoff potential resulting from the use of mechanistic methods of analysis in the design process; and
3. Evaluation of the effect on the potential payoff of the use of mechanistic methods with regard to variations of the methods of analysis and variations in the models used and in parameters that influence behavior.

Needed research with long-term goals includes the following:

1. Consideration of stochastic variables;
2. Consideration of more complex (and perhaps more realistic) models of material behavior, including continuum and particulate mechanics problems; and
3. Study of fracture mechanics.

It is recognized that much work is under way in all these fields. The work, however, is not organized to assist in the solution of problems directed toward asphalt concrete pavements. It is hoped that, by publicizing the need and challenge, competent researchers will direct their attention to these problems.

DISCUSSION OF SOLUTIONS

There are two major points of discussion germane to the solution of boundary value problems. First, the relationship of boundary value problem solutions to the overall picture of pavement design is of concern. Second, the techniques that may be employed in solving a given boundary value problem are of direct concern inasmuch as they control the types of problems that can be solved.

Relationship to the Design Process

The design community will not accept a boundary value problem solution unless there is convincing evidence that such a solution will provide an improvement to current practice. Improvement can be defined in many ways. One criterion for improvement is economics; a larger benefit (also to be defined) for a smaller cost. Another criterion is that a boundary value problem solution provides predictability where other methods do not. A third criterion for improvement is that a boundary value problem solution lead to the design of a longer lived pavement without increased cost. A fourth criterion is an increased margin of highway safety.

In addition to these criteria, there is a belief by some engineers that present design procedures are vague, are inconsistent, and lack credibility. A design method based on the solution to one or more boundary value problems will, by its consistency, provide a rational basis for extrapolation and for analysis of field observations.

If a design method based on boundary value problem solutions is proposed for adoption, it must prove itself under prototype conditions. A systematic and well-documented field measurement program is thus requisite for the final adoption of the proposed method.

The solution of boundary value problems is a single element in a total pavement design system. It is strongly suggested that attention be directed to the total system and its requirements as a prerequisite to the investigation of solutions to boundary value problems.

Problem-Solving Techniques

The present state of knowledge and technology is such that most relevant boundary value problems in linear elasticity or viscoelasticity are capable of solution by numerical methods. It is noted, however, that many problems involving more than two dimensions are not capable of economic solution. The size of computer and the amount of machine time required are much too large.

Unfortunately, a considerable amount of naiveté exists concerning the state of operational programs. Most of the programs referred to are operational only in a limited sense. Documentation is generally poor to nonexistent; maintenance does not exist. If boundary value problem solutions are to be useful, there must be an organized, before-the-fact development of documentation and maintenance procedures. There also must be recognition of the fact of unbundling and the existence of useful proprietary programs.

There are two types of computing systems that are of concern: (a) those programs designed to solve given boundary value problems and usually (but not always) characterized by relatively simple data bases accompanied by complex computational algorithms, and (b) data management systems, which are usually (but not always) characterized by large data bases (both numeric and nonnumeric) and by relatively simple algorithms. A pavement management system will certainly encompass both types. Thus, any consideration of computer and computing techniques must be concerned with both data structuring and algorithm development.

There are three factors of concern in the development of a computational system: machine compatibility, program documentation, and program maintenance.

Machine compatibility is a function of the computer language. If a system is written in Assembly Language, there is little hope that conversion can be made from one machine type to another without complete reprogramming. If a system is designed to be machine- or vendor-independent, the system must be written in FORTRAN or COBOL. Even under these conditions machine independence is a myth. USASI FORTRAN IV is such a limited subset of FORTRAN IV as to be relatively useless. Beyond this, different machines have different characteristics. For the CDC 6000 series machines, FORTRAN IV is a superset of FORTRAN IV for IBM 360 machines. A downward conversion is a nontrivial task. The reverse is true for COBOL.

Recent efforts in computer languages have centered around the development of macroprocessing languages. These languages are mobile in that they can be moved from machine to machine. They also have the machine efficiencies of assembly language.

Adequate documentation of even a moderately complex system should have the following four components:

1. An analysis and algorithm manual that provides a detailed exposition of the analysis used and the computational algorithms.
2. A user's manual that provides detailed instructions on the use of the system and that contains a set of example problems with results providing the user with check results when he sets the system up on machines other than the prototype computer.

3. A reference manual that includes a fully commented, machine-generated listing of the system and that contains write-ups of each subsystem, subprogram, and subroutine as well as a complete dictionary of all variables (macro and micro flow charts of the system and its components are essential).
4. An engineering manual that provides the engineer with a number of examples of the use of the system and that combines the problem-solving capabilities of the system with its field use.

The machine-generated listing mentioned with the reference manual should not be a straight listing but should be a system printout generated with a successful run. There is no such thing as a machine-independent language. Consequently, the reference manual should contain information on the hardware used, the language, the operating system, storage requirements (both central memory and peripherals), and other machine characteristics of importance.

No large- or even moderate-sized computational system is free of programming errors (bugs). Furthermore, the frequent changes in operating systems often necessitate changes in the computational system. Thus, there is a long-range responsibility in maintaining the computational system. The maintenance of a system is something more than a technical problem. Unless a maintenance responsibility is clearly understood and defined at the outset, many serious areas of dispute can occur.

There is an increasing reliance on computer programs in the pavement design process. Research and development efforts in this area will have, as an end product, a computer-based system. It is thus required that any computer programs that are developed have two major attributes:

1. Portability—the capability of moving a program from one machine environment to another with a minimum of effort; and
2. Adaptability—the capability of facile modification of a system to relate to specific user needs. For example, a program designed for a UNIVAC 1108 should be designed with the possibility that a subset of the program may be adapted for an IBM 1130. Also, programs originally designed for batch process operation should be adaptable to time-sharing.

The development of techniques for portability and adaptability is a currently active research area in computer science and technology. These efforts, to date, have shown that the initial construction of a portable, adaptable system must clearly separate the input/output structure, the algorithms, and the data structure. For example, an algorithmically based system is potentially portable if the algorithm is embedded in the input/output structure. Portability is not feasible if the input/output structure is embedded in the algorithm.

In addition to program development, it is essential that program maintenance principles be established. Investigators who develop programs are generally not inclined to maintain a system once it is released. It is a tedious, technically unrewarding task. On the other hand, an unmaintained system will very shortly pass out of existence because of the inability of the user to make the system work. To avoid future problems requires that a maintenance plan be clearly established at the outset of the programming effort.

One of the established efforts within the Federal Highway Administration is TIES (total integrated engineering system). This is a massive effort dedicated to the development, use, and maintenance of computer programs for highway engineering. It is recommended that the computer systems aspects of asphalt concrete pavement systems be developed within the concepts of the TIES program.

RELEVANCY OF SOLUTIONS

The solutions to boundary value problems are relevant to the pavement design problem as long as it can be demonstrated that they provide an improvement to the design process, as was discussed previously. Questions of usefulness and economy are also dependent on improvement. In this case the degree of improvement is a governing criterion. It is not clear that this question has been, or even can be, answered directly.

There is no evidence in the field of pavements. There is, however, ample evidence in other engineering disciplines that significant improvements can be and have been achieved by "upgrading" the design process by use of "higher level" boundary value problem solutions.

When the question of relevancy is viewed in its broader context, it is legitimate to be concerned with the relationship of pavement design improvement to other national efforts. An effort in this direction, at this time, may well divert critically needed manpower and other resources from projects of higher priority. On the other hand, an effort of this nature may be within the national goals of our society. This is a higher order question and concerns the types and levels of national priorities.

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GROUP C

DESIGN CONSIDERATIONS

Chairman, James F. Shook; recorder, John W. Hewett; members, Arthur T. Bergan, W. Ronald Hudson, D. A. Kasianchuk, James W. Lyon, Jr., Thurmul F. McMahon, Lionel T. Murray, George B. Sherman, Bernard A. Vallerga, and S. R. Yoder

The objective of Group C was taken to be that of describing the entire pavement design process. The process was considered to be largely one of making decisions (based on structural or engineering, economic, or other considerations, some of which can be described as matters of policy) by using a system similar to that shown in Figure 3 in the paper by Hudson. However, the group members did not unanimously support this objective. Some members felt that the design engineer is not concerned with the entire design process but is concerned only with that part of it that considers traffic, soil strength, and present serviceability levels. In the somewhat lengthy discussions, some agreement was reached on this matter, but, for the most part, the disagreement was not resolved. It is hoped that these differences of opinion are reflected in what follows.

SYSTEM DESCRIPTION

A simple description of the pavement design process is shown in schematic form in Figure 1. This form was not discussed specifically by the group, but it seems to meet the needs of most members. It is offered with the knowledge that considerable work needs to be done before it can be called a working framework on which to build an acceptable design system. A number of investigators have attempted to formulate a framework for subsystems, as suggested by the references to the papers in Part III. The group in general agreed that desirable input variables are those described in box 2 of Figure 1. There was considerable discussion, however, as to how much detail should be considered. In particular, the need to predict the variables of traffic and materials was impressed on the group by those who now actually design pavements.

There was less agreement on desirable output. However, in one way or another, group members wanted the design system to give them the capability of designing to prevent cracking, for example, as well as PSI, and the capability of dealing directly with the primary output variables, if they so desired.

There seemed to be some confusion as to the role of economic and other "soft" design criteria and a reluctance at first to admit them into the system. Perhaps these should have been treated as constraints on the system rather than criteria. It is suggested in the schematic that they might be either or both.

SOME NEEDS AND PRIORITIES

Needs and priorities were discussed and agreed on by the group. High priority was given to only three items:

1. Techniques need to be developed, preferably from current knowledge, for transforming the input variables (box 2, Fig. 1) into the primary output variables (box 3). Group members felt that the capability of predicting states of stress and the like was needed by the design engineer who might not be too concerned about how the stress, for example, was calculated but who would be concerned with making use of these calculated stresses. For this reason, deflection was left as a desirable output variable. It was recognized that separate subsystems might be needed, considering the present status of knowledge, for each primary output response variable. Potential use of the system for

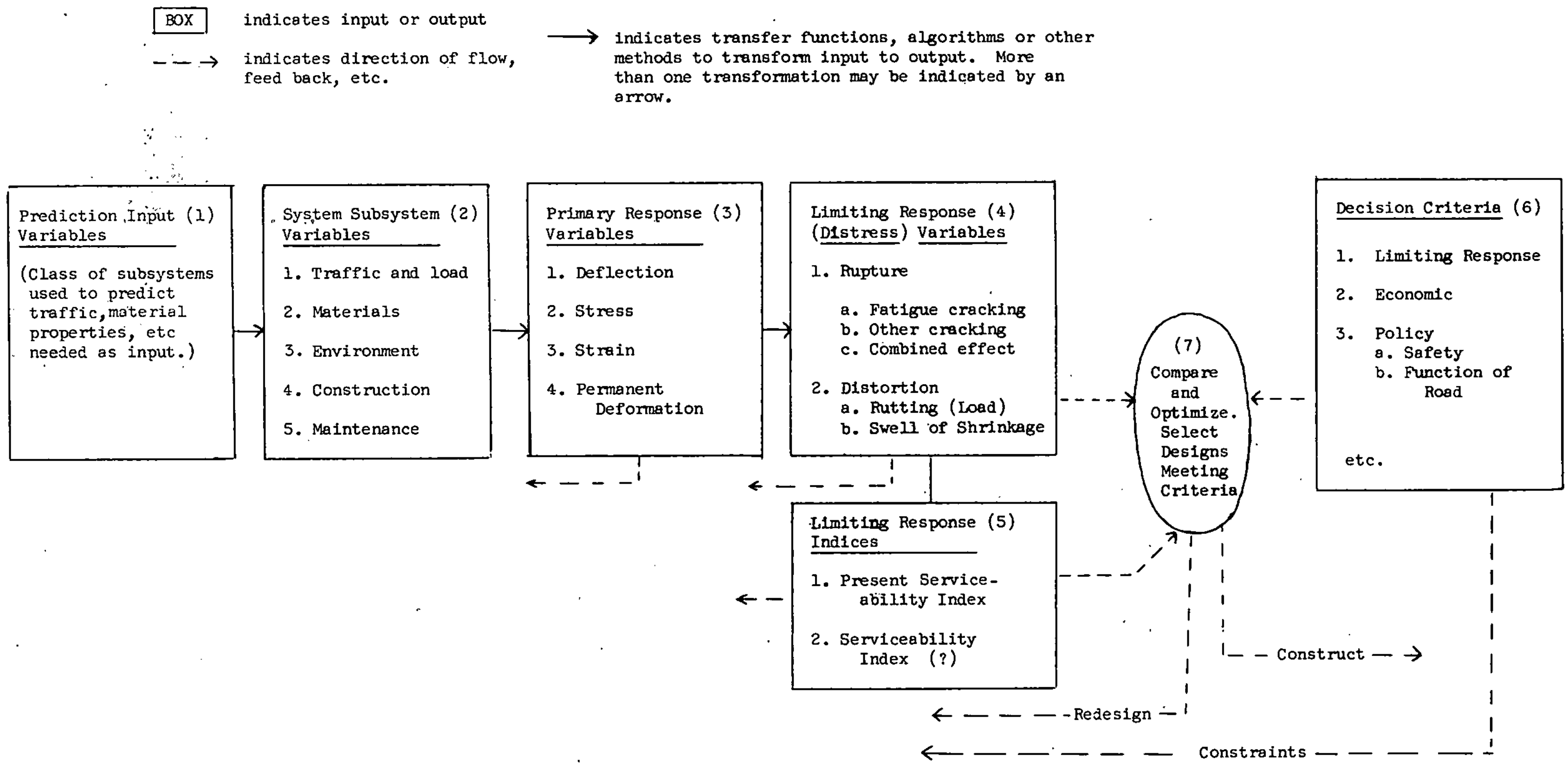

Figure 1. Schematic of pavement design system.

overlay design was mentioned by some group members. It was felt that the designer should exercise as much control and as many options as possible at any major step in this phase of the design process. The group felt that technology is available for constructing at least a primitive form of the required transfer functions.

2. Techniques are needed that will allow the designer to predict the limiting response variables (box 4) from the primary response variables (box 3). It was implied that the designer may want to exercise control over this step in the process. For example, he may want the capability of relating strain directly to fatigue life or, if possible, of predicting the quantity of cracking that he feels represents a desirable end point. (In a related manner, he also would like the option of selecting a desirable level of rutting and PSI.) It was felt by the group that developing at least a primitive subsystem to accomplish this need is within the capabilities of currently available knowledge.

3. One problem with input variables was selected as a high priority need. It was felt that many times it is impossible (a) to characterize material properties as they actually will exist in the field, or (b) to predict effects of environmental changes on the condition of the entire pavement system. This is particularly a problem with subgrade soils where environmental changes often occur after construction. High priority should be given to this problem as it relates to soils. Present technology is weak, and little effort is being made to accelerate such studies.

Lower priorities were assigned to many other elements in the system, either because they were less critical, because sufficient progress was already being made in the area, or because sufficient means were available to effect an interim solution if the necessary effort were put forth. Some of the items discussed follow.

1. Traffic and load variables—Better prediction techniques and the capability of handling new tires, increased tire pressures, loads, and so forth are necessary.
2. Materials variables—Better methods for predicting characteristics that actually represent field conditions, more reliable methods for characterizing materials, and a better understanding of effects of variability on design life are needed. Some capability in the last area will be available through sensitivity analyses using present knowledge of materials and testing variability and the design system if it is developed.
3. Environment—Never completely defined, this item was recognized as important in that environmental changes affect material properties. It was felt that temperature prediction techniques were well along in development but that moisture prediction techniques were not. It was not decided whether moisture or some other measure such as soil suction was really the prime variable. Study is clearly needed in this area.
4. Construction variables—Study is needed in two main areas. The group felt that it is very difficult to be sure that we get in the field the property or condition needed for optimum performance. Also, the effects of variations in construction operations need to be evaluated. Here the group felt that, once a design system is available, the necessary sensitivity analyses could be performed relatively easily by using currently available knowledge of construction material variability.
5. Maintenance variables—Maintenance variables were recognized as valid input but were not defined or discussed. This needs to be done.
6. Safety—Study needs to be initiated into methods for quantifying relationships between decision criteria such as safety and limiting response variables such as rutting.

The group chairman recommends that immediate steps be initiated by FHWA to synthesize available knowledge into a working model of the pavement design system by using as much as possible of the knowledge discussed at this workshop, that this system be made available to designers and researchers, and that study and use of the system be encouraged to determine whether we are on the right track. This step might show that many elements in the system now considered to be important may not be important, and many considered to be unimportant may be more important. Some steps thought impossible may prove to be possible, and vice versa. We should stop looking at pieces of the puzzle, start putting the puzzle together, and see what the picture looks like.

GROUP D

LOAD AND ENVIRONMENTAL VARIABLES

Chairman, Eugene L. Skok, Jr.; recorder, Matthew W. Witczak; members, James Brown, Ralph C. G. Haas, James H. Havens, James M. Hoover, Wallace J. Liddle, Chester McDowell, Aleksandar S. Vesic, E. B. Wilkins, Stuart Williams, and Eldon J. Yoder

Group D felt that each of the variables being considered is predictive in nature and represents the extreme variability of conditions throughout the design life of a pavement. It is, therefore, necessary to obtain as many data as are economically feasible and to be able to judge how accurate that information is. Other groups that will be using this information should consider how accurately the variables of load, temperature, and moisture content need to be determined.

LOAD EFFECTS

For evaluating load effect, the best system available is the equivalent axle load concept. For highway pavements, this type of representation can be used to predict the load effect on the pavement in terms of fatigue distress or permanent deformation. It does not, generally, make it possible to estimate the ultimate load capacity of a pavement in terms of shear failure.

Use of the Equivalent Load Concept

The method of dividing the traffic into various vehicle types to study the effect of an individual type is considered appropriate. When the effect of each vehicle type has been established, this value is multiplied by the number of vehicles. These effects are summed for all vehicles and then summed for all years during the design period to give the total equivalent number of axle loads. The load equivalency factors from the AASHO Road Test are considered the most appropriate to use at this time because they are based on the only data now available on the effect of axle loads on pavement performance.

Methods have been presented for using the AASHO Road Test equations directly for estimating the load effect of the various axle loads and configurations used at the road test (1, 2, 3). The load effects thus determined are called the load equivalency factors.

To apply the equivalent load concept to actual traffic situations, it is necessary to determine the number and weight distribution of the axles to use a roadway during the design period. The appropriate load equivalency factors are then multiplied by the number in predetermined weight classes and summed to give the total number of equivalent loads. A number of methods that use these concepts have been developed and presented by various authors (4, 5, 6, 7, 8).

The following is an example of a method that can be used to estimate total equivalent loads by using the present state of the art.

1. A determination should be made of the average effect of various types of vehicles based on the AASHO equivalency factors. This can be done on a statewide basis by using standard W-4 tables that are available each year. The W-4 tables present a summary of the distribution of axle weights for each state, as well as the average equivalent load effect for each type of vehicle. The distributions in the W-4 tables represent the statewide data for a given year and state. For a particular location that is not felt to be representative of statewide conditions, the individual factors should be determined (6). Information on related analyses is available from the California and Utah highway departments or in other works (4, 5, 6, 7, 8).

2. The variation of the average vehicle effect (N18 factor) from year to year must be estimated. This can be done by using W-4 tables of previous years and considering the annual change in the factor. This change may then be projected into the future with some limit put on the maximum value that can be attained. Some work of this nature has been done by the Utah and Kentucky highway departments and by the University of Minnesota.

3. The number of each vehicle type used for the analysis must be determined for the design period. This distriubtion of vehicles may vary significantly throughout the period and should be considered. This information can be obtained from the traffic section of most state highway departments. The distribution is usually based on the type and use of the highway being planned.

4. The N18 factors are multiplied by the number of respective vehicle types and summed to give the total number of equivalent axle loads.

Other representations of traffic, such as maximum wheel load, average of 10 heaviest wheels daily, and so forth, are now being used. The method outlined represents what is considered an example of the present usable state of the art with respect to the equivalent load concept. For low-traffic roads the equivalent load procedures should be modified to accommodate a limited number of relatively high axle loads. The mode of failure for this situation would not be fatigue but would be based more on an ultimate load concept.

Needed Short-Term Research

First of all, to establish the error involved in the present system of estimating equivalent wheel loads, we should establish the accuracy of present predictions. This can be done in a manner similar to that done by the Kentucky Department of Highways and by the Franklin Institute Research Laboratories (10).

Load equivalency factors should be established for heavier axle loads and for different axle load configurations. It would be best if this could be done by observing the effects of various axle loads on the distress in a pavement section under controlled conditions, but this is a costly process. It is, therefore, recommended that these other configurations be evaluated by using a calculable stress or strain that can be used to predict a given mode of failure.

A functional road use system should be established to categorize highways in order to determine appropriate load effects and vehicle type distributions. This type of road use classification would make it possible to consider the type of traffic along with the volume of traffic. With this information it would be possible for highway departments to utilize the system more readily. The research should be directed toward determining how accurately the traffic parameter needs to be known for design purposes.

Future Research Needs

Load equivalency factors should be determined for each mode of distress. These will be dependent on the stress or the strain or deflection or both used to evaluate the pavement failure mode. Examples of such modes are the calculation of tensile strain in the bottom of the asphalt layer to minimize cracking and the calculation of vertical strain at the subgrade to minimize permanent deformation.

The effect of vehicle speed on the equivalent axle load factors should be established. This effect would consider the additional effect due to impact loading on the pavement and the decreased effect at higher speeds due to the inertia of the pavement section. The dynamic load effect has been studied at the University of Texas by using weighing equipment developed there (9, 10).

As mentioned earlier, more weight and volume data are needed so that the significant parameters can be estimated more accurately. This can be done by making more weighings and by continuous weighings either by using portable scales at various locations or by using an electronic weighing device such as that developed at the University of Texas.

ENVIRONMENTAL VARIABLES

At the present time the environment is being evaluated in a number of ways. Design procedures generally use one of the following concepts: (a) the most critical condition, (b) a regional factor, or (c) the variation of in situ conditions. Each of these in some respect relates to the temperature and moisture conditions estimated for the pavement section during its design period. The presence or absence of frost and the depth of frost are also implied in the evaluation of environment.

TEMPERATURE VARIABLES

Tables were constructed to aid the group in considering the input of the temperature and moisture variables into the evaluation of performance with respect to environment. The group checked those items that it unanimously agreed could have an effect, although some felt that more intensive study would indicate many of these to be insignificant. An explanation of each check is not given here because the effects of temperature or moisture content or both have been discussed by Group A.

Table 1 gives the temperature factors that affect the respective layers of the pavement section. Where the pavement sections and temperature functions coincide (indicated by "X") there is an effect of that temperature function on that layer; thus, the number of Xs generally indicates how important that variable is. The effect of temperature on the properties of soils has been presented elsewhere (20).

At the present time the level of air temperature at an hourly basis is available at almost any location. In some cases this has been synthesized and is being used to estimate the properties of the asphalt concrete. The air temperature has been correlated to pavement temperatures for various conditions of wind and cloud cover by a number of groups around the world. This information is available in various Highway Research Board publications (11, 12, 13, 14) and in other journals. The gradient of temperature through the pavement section has been determined by The Asphalt Institute, the Kentucky Department of Highways, the AASHO Road Test researchers, and others (12, 15).

The daily and monthly variations of temperature at a number of locations have been used by The Asphalt Institute to predict monthly average temperatures over a design period (16, 19). Daily variations of temperature have also been used at the Ste. Anne Test Road, Hybla Valley, the WASHO and AASHO Road Tests, The Asphalt Institute, West Virginia, and other tests (13, 14, 17, 18, 19, 26).

The use of heat transfer equations to predict the gradient and level of temperature throughout a pavement section has also been done on a limited scale. Solutions to these equations are available at Purdue and other locations.

Short-Term and Long-Term Needs for Temperature Variables

For both the short-term and long-term it is recommended that the present temperature data be expanded, that more be gathered, and that they be synthesized into the

TABLE 1

EFFECT OF TEMPERATURE FUNCTION WITH TIME ON ASPHALT PAVEMENT SECTION LAYERS

Temperature Function	Asphalt Concrete	Base				Subgrade
		Granular	Asphalt-Treated	Cement-Treated	Lime-Treated	
Level throughout layer (mean, max, min)	X	X[a]	X	X	X	X[a]
Gradient through section at given time	X	–	X	?	?	X
Cyclic						
Long-term	X	–	X	X	X	–
Daily	X	–	X	–	–	–
Space variation (longitundinal and lateral)	X	X	X	X	X	

[a]Frozen.

TABLE 2

EFFECT OF MOISTURE CONTENT WITH TIME ON ASPHALT PAVEMENT SECTION LAYERS

Moisture Content	Asphalt Concrete	Base				Subgrade
		Granular	Asphalt-Treated	Cement-Treated	Lime-Treated	
Level throughout layer (mean, max, min)	–[a]	X	X	X	X	X
Cyclic (long-term)	–[a]	X	–	X	X	X
Time and space	–[a]	X	X	X	X	X

[a]Moisture effects are unknown at this time, and the group felt them to be insignificant.

forms found significant for the particular failure mode being considered. For instance, at The Asphalt Institute it was found that the monthly level of temperature could be used to reasonably estimate the level of temperature in the asphalt surface for a fatigue analysis of an airport design (16). Similar analyses should be made on the effects on other failure modes for both airfield and highway pavements.

Moisture Variables

Table 2 gives the moisture functions that affect the layers of an asphalt pavement. Again, the areas with Xs indicate that there is an effect of that moisture function on that layer; thus, the number of Xs generally indicates how important that variable is.

At the present time there are essentially no moisture data available in published form for asphalt-and cement-treated bases. For granular bases and subgrades, there is information published by the Highway Research Board (20). There is, also, information from Australia, South Africa, and other areas. For both the short-term and long-term, this information needs to by synthesized and related to local factors such as rainfall, humidity, topography, drainage conditions, soil properties, and pavement section characteristics (10). For the long-term other similar tests should be run in representative areas throughout the country to improve the correlations. Very little information is published on cyclic variations in moisture contents.

In Oklahoma a study has been made in which the variation in moisture content of embankment soils under pavements with time has been determined (21). It is recommended that more studies be made and that cyclic variations be determined, especially for periods of precipitation and drying. These variations should also be determined for bases and subgrades both for general moisture changes and especially for moisture levels during the spring thaw. The moisture content at the interface between thawed and frozen materials should especially be determined. These conditions should be determined in a few areas where performance and materials properties are being monitored with time to see whether these moisture conditions have a significant effect on pavement performance.

The distribution of moisture longitudinally and laterally has been studied at the British Road Research Laboratory by researchers whose findings are presented in Highway Research Board reports and at the AASHO and WASHO Road Tests (13, 14, 15, 18, 22, 23, 24). This variation should be checked at specific test locations where performance and materials properties are being monitored to see if there is a significant effect of variation in moisture content.

SUMMARY

In this report an attempt has been made to summarize the state of the art in load, temperature, and moisture variables that affect the performance of an asphalt pavement. Some references have been given as examples of this work, but it is realized that much more work has been done in these areas. We apologize for the omissions, but time does not make it possible to review and present all of the work being done. Perhaps in the near future it will be possible to make a more complete study of the current work.

It is suggested that the equivalent load concept is the best method to use for evaluating traffic and that research be directed toward improved procedures for collecting and using traffic volume and weight data for this purpose. The accuracy of the final equivalent loads calculated should be determined. In addition, a determination should be made of how accurate the parameters should be for practical design purposes.

A few methods have been established that use temperature information directly to sum up the fatigue effect on an asphalt pavement. Studies should be made for both airport and highway pavements to define and establish the temperature effects more accurately and to determine the degree of accuracy necessary for a reasonable pavement design.

The effect of moisture level and its function with time on the performance of a pavement have also been considered. Work should be directed toward developing an accurate function to predict moisture content in embankments and granular bases with time so that the variation in materials properties with time can be determined. The required accuracy for a practical design situation should also be determined.

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GROUP E

TRAFFIC-INDUCED FRACTURE

Chairman, Donald A. Kasianchuk; recorder, K. Majidzadeh; members, James Brown, John A. Deacon, B. F. McCullough, Thurmul F. McMahon, Keshavan Nair, George B. Sherman, and Matthew W. Witczak

VERTICAL LOADING

Repeated Applications

For fractures induced by vertical loading, the phenomenological state of the art is sufficient and capable of introduction to practice. The block diagram of a fatigue subsystem shown in Figure 1 provides a possible framework for implementation. It is believed that a more detailed mechanistic approach, based on fracture mechanics principles, is needed for analysis of fatigue. Such an approach is achievable in the near future and could be introduced into practice.

Needed Research: Phenomenological Approach—The phenomenological approach to fatigue is that one developed by Monismith, Deacon, Pell, and others. The state of the art is briefly summarized by Deacon in Part III of this Special Report. The proposed design subsystem includes the use of the results of fatigue testing of asphalt concrete beam specimens, the characterization of materials of the pavement section by using cyclic load triaxial testing within the environmental and load ranges expected during the service life of the pavement, and the prediction of stresses by using the available solution to the layered, two-dimensional, elastostatic boundary value problem. This method, it is claimed, has the capability of predicting the time to first cracking of the asphalt concrete layer caused by repeated traffic loadings.

The method has been used in several instances, both for analyses of existing in-service pavements and for the design of proposed pavement facilities.

To develop an accepted design method requires the following:

1. Verification—If it is felt that a method is capable of introduction into practice, no further research is indicated except that necessary to show that the method works. This can be done by using the subsystem to explain known occurrences of fatigue distress or by using the subsystem to monitor the behavior of designed pavements.
2. Sensitivity analysis—Many of the items considered at this workshop directly affect the fatigue subsystem. The function of sensitivity analyses proposed is to determine the level of effect on the fatigue life prediction of variation in the inputs to the subsystem. This kind of information is required by the user of the subsystem in allocating his efforts in the acquisition of the data required by the process.

In the long term, to develop a more acceptable, reliable design method requires the following:

1. Verification—Verification functions here in the same manner as for the short term.
2. Knowledge of mode of loading effects—The applicability of laboratory fatigue test results to fatigue life prediction of real pavements requires knowledge of the mode of loading (ranging from controlled stress to controlled strain) that exists in the field.

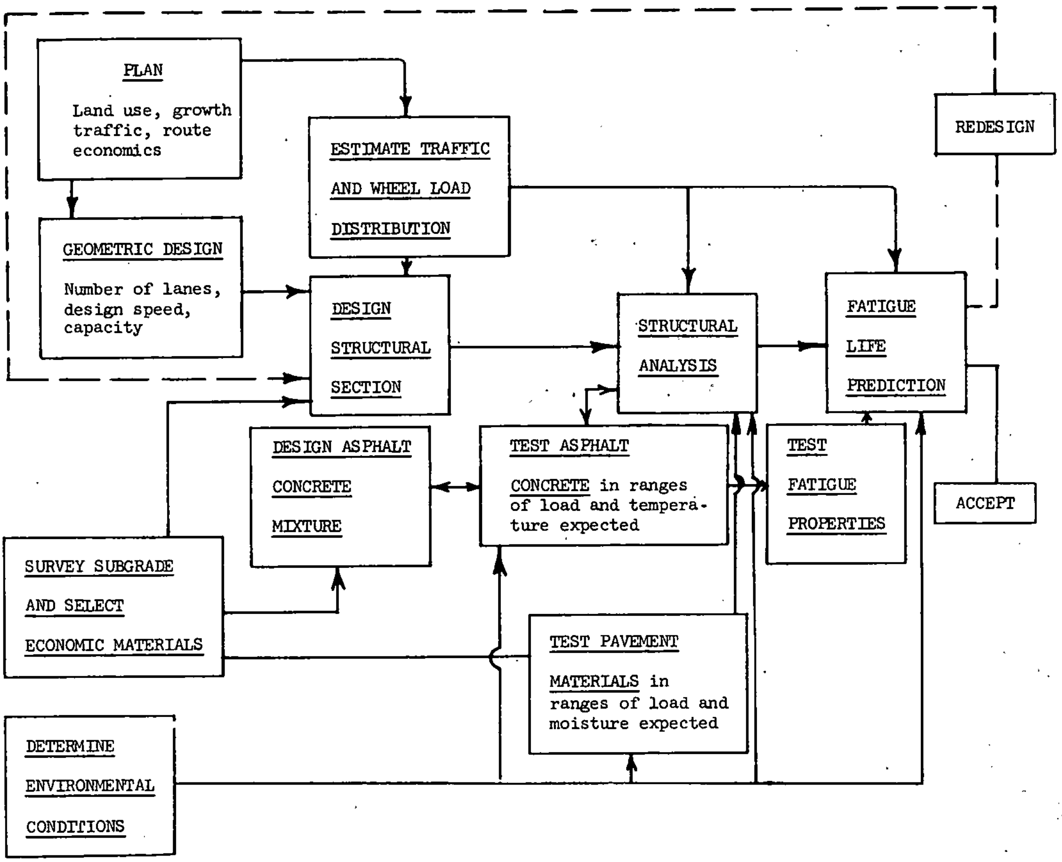

Figure 1. Fatigue subsystem.

3. Development of a test method—The reduction of the laboratory effort required to produce a fatigue curve for a specific asphalt concrete is desirable. Efforts are under way in this area, but more verification is required.

4. Understanding of areal extent of first cracking—The difficult problem of what happens after the first crack has appeared in the pavement must eventually be solved. The nebulous term "percent cracking" must be defined and quantified and methods provided for its prediction.

5. Combination of temperature and loading—An examination is suggested of the possibility of "thermal fatigue" being additive to traffic-induced fatigue.

6. Use of stochastic models—The variability of every input to the subsystem, along with the uncertainty inherent at the design stage, requires that techniques be provided to allow the designer to consider their effects on his decisions.

Needed Research: Mechanistic Approach—The mechanistic approach used the fracture mechanics principles for crack initiation and growth mentioned in Moavenzadeh's paper in Part III and developed in a forthcoming AAPT paper by Majidzadeh. The effect of geometry, boundary conditions, and load time history are introduced in terms of a stress intensity factor. The model is also capable of considering variability in the material properties. Further research is needed in this area, specifically to incorporate random loading effects, localized elastoplastic effects due to excessive loading, and accumulation of cracking for performance analysis.

To develop an acceptable design method requires verification (data acquired in attempting to satisfy the need for verification of the phenomenological approach also serve for mechanistic approach), localized plastic flow-elastoplastic interaction, and random loading.

In the long term, to develop a more acceptable, reliable design method requires verification, combination of load effect and temperature effect, and stochastic loading.

As mentioned earlier, the mechanistic approach is not so well developed as is the phenomenological one. On the basis of a brief outline of its theory, Group E was impressed with the following possibilities for this approach:

1. Mode of loading effects are handled through the stress intensity factor that is calculated for both the laboratory test and the field condition;
2. The output appears in terms of areal extent of cracking; and
3. The method is based on a stochastic notion, that of the distribution of flaws within the asphalt concrete layer and, thus, must proceed stochastically.

At this point, we are faced with the problem of making a decision in the face of uncertainty. Until further work is done with the mechanistic approach, we are unable to assess the degrees to which its possibilities will be attained. At the same time, until we can assess the degree to which these possibilities can be attained, we are unable to firmly define what further work is required.

It should be noted that the mechanistic approach and the phenomenological approach pertain only to the blocks in Figure 1 labeled "test fatigue properties" and "fatigue life prediction." The considerations included in the other blocks are common information in both approaches. The verification work discussed earlier can serve its duality of purpose and should be carried out.

Single Application

In this category of fracture induced by traffic loading fall the considerations of the effects of abnormally large loads. At present we would attempt to ascertain the effects of one application of such a large load through either the bearing capacity approach such as that of McLeod or a punching shear approach such as that of Meyerhof. Some possibility exists for the development of the mechanistic approach to crack initiation and propagation discussed earlier, and this should approach a solution to this problem. (No research is suggested in this area.)

HORIZONTAL LOADING

The type of fracture induced by accelerating or decelerating vehicles or by articulated vehicles with a capability of rotating a single wheel around a vertical axis seems to be a problem of low priority in highway pavements of good construction. Problems associated with poor bond between the surface and the immediately underlying layer, however, can occur. This problem may be of more interest in airfield and other special paving situations (e.g., dock facilities for containerized cargo harbors). We suggest that some boundary value problem solutions exist for this problem, but some research should be initiated toward description of the failure mechanism.

OTHER RESEARCH TOPICS

Fatigue of Treated Material Layers in the Pavement Section

Because of limited time, Group E was not able to discuss an admittedly important topic, fatigue of treated material layers in the pavement section. Some information regarding the fatigue of asphalt-treated bases might be inferred from the accumulated knowledge regarding the fatigue of asphalt concrete. In the case of cement-treated bases, there does exist a fatigue-based design method for the selection of thickness of a layer of soil cement that meets the specifications of the Portland Cement Association.

Reflection Cracking

A most urgent need for research at this time is associated with reflection cracking. Although the problem is best known as associated with overlay design, the fracture of an asphalt concrete pavement due to the reflection of a crack in a cement-treated base is at least one example of its occurrence in new construction.

The need to develop a model that can explain the failure mechanism seems best measured by our inability to decide whether reflection cracking is or is not a traffic-induced fracture. Although we can accept that volume changes in an underlying material layer can contribute to the development of reflection cracking, we are uncertain what role is played by repeated traffic loading either in a flexural way or in a relative vertical movement way. This is considered an urgent research need.

CONCLUSIONS

The following items are of roughly equivalent priority and relevance to the development of design methods to preclude traffic-induced fractures:

1. The development of a model of the mechanism of reflection cracking;
2. The verification of the statement that there now exists a method for the design of asphalt concrete pavements; and
3. The development to the design stage of the mechanistic approach to fracture under repeated loading.

GROUP F

OTHER FRACTURE

Chairman, E. B. Wilkins; members, Milton E. Harr, Richard A. McComb, Wolfgang G. Knauss, Phillip L. Melville, Eugene L. Skok, Jr., Bernard A. Vallerga, and Russell A. Westmann

The basic purpose of this session was to assess the current state of knowledge and available design methodology relating to "other" (non-traffic-load associated) fracture of flexible pavements. A subsequent purpose was to identify the deficiencies in this methodology and to recommend both short- and long-term research and development needs.

It was agreed initially that the forms of other fracture that are of concern are:

1. Shrinkage cracking (i. e., that associated with thermal cycling, aging, aggregate absorption, and moisture changes);
2. Thermal contraction cracking;
3. Reflection cracking (i. e., existing cracks reflecting up through overlays or surface treatments);
4. Foundation-associated cracking (i. e., settlement, heave, and freezing shrinkage); and
5. Construction-associated cracking (i. e., overrolling) in asphaltic concrete surfacing.

It was further agreed that the type of other fracture that was of concern was that which manifested itself in cracking of the surface.

Finally, Group F agreed to concentrate its deliberations on items 1, 2, and 3 for several reasons: (a) There was a greater possibility of short-term solutions for these three, and (b) it appeared more logical to consider the situations where fracture is in itself the primary mechanism. (See the Group H report for a discussion of item 3.)

The following conclusions were reached by Group F. (See Table 1 for recommendations.)

1. Other fracture can occur or be initiated in any component of the pavement. However, the most prevalent is one that is initiated in the bituminous surface, except for the case of reflection cracking.
2. The effects of other fracture do not usually initially impair serviceability. However, because of moisture intrusion, loss of load transfer, and the like, accelerated deterioration can occur. Moreover, cracking is unsightly to the traveling public, the effect often being accentuated by poor maintenance practice, and indicates certain deficiencies in the structure. The combined effects often result in premature resurfacing and additional costs.
3. Shrinkage or thermal contraction cracking or both are prevalent to varying degrees throughout most of the United States and Canada. However, the causes can vary from region to region and are not always readily apparent. For example, in northern areas extreme low temperatures have been shown as the dominant cause. In southern areas asphalt aging combined with termal cycling may be important, although conclusive proof for this does not exist.
4. A working subsystem (1) to eliminate or to predict the occurrence of thermal contraction cracking in new pavements is available. This is based on extensive field (including several road tests) and laboratory investigations.

TABLE 1

RECOMMENDATIONS FOR RESEARCH

Recommendation	Ranking	State of Art	Priority: Short Range	Priority: Long Range
Develop a mechanistic model of "reflection cracking" for (a) non-load-associated conditions and (b) stress-strain conditions	1	Model needed	1	1
Evaluate limiting stress guides to control thermal contraction cracking for a wider range of environments (i.e., refine)	2	Model available	—	1
Develop a prediction model having cracking index as dependent variable for various regions by using regression methods and available data	3	Some regressions available	2	2
Develop predictive procedure for subgrade-initiated shrinkage cracking (i.e., due to freezing shrinkage)	4	Some work in Alberta		3
Evaluate thermal cycling combined with aging effects on shrinkage factors	5	Model needed	—	3
Evaluate effects of varying maintenance strategies for non-load cracking in relation to serviceability	6	Some rough data available (CGRA)	X	X

5. The main component of the pavement considered in the working subsystem is the bituminous one. It is best characterized by stiffness (van der Poel definition). The method of determining stiffness can be either direct (Haas and Anderson methods) or indirect (Shell method). Additional work is required to correlate the two methods for bituminous materials in America to supplement the Shell research in Europe.

6. Reflection cracking is a serious problem (or will be in many areas unless better design and control methods are developed) in many parts of the United States and Canada because of the many miles of existing cracked pavements. Reflection cracking can be delayed (up to 2, 3, or more years) by "sandwiched" granular layers or by thick overlays, but it is difficult to eliminate. It must be recognized, though, that reflection cracking may in many cases be primarily load-associated.

7. Thermal contraction cracking occurs in Canada when the stiffness modulus of the surfacing exceeds approximately 2×10^6 psi at the lowest temperature occurring in the surfacing. This corresponds to a stiffness of approximately 3×10^4 psi for the bitumen used in a dense-graded paving mixture compacted to 3 percent air voids. At this stage little is known concerning the prediction of low-temperature stiffness variations during the aging process.

The group suggests as a guide the following working system of limiting stiffness:

Minimum Expected Temperature (deg F)	Stiffness Modulus (psi): Cracking Expected	Stiffness Modulus (psi): Cracking Eliminated
-40	1,000,000	500,000
-25	700,000	300,000
-10	400,000	200,000
+10	100,000	50,000

REFERENCE

1. Haas, R., et al. Low Temperature Pavement Cracking in Canada: The Problem and Its Treatment. CGRA, 1970.

GROUP G

TRAFFIC-INDUCED PERMANENT DEFORMATION

Chairman, Ronald L. Terrel; reporter, N. K. Vaswani; members, Ernest J. Barenberg, James H. Havens, John W. Hewett, James M. Hoover, Fred Moavenzadeh, and Harry A. Smith

Permanent deformation due to moving traffic can be defined or at least limited to time-dependent distortion or volume change or both caused by densification of one or more layers within the pavement system. Deformation can take place in one or all layers although it is noticeable only at the surface in the form of ruts, lateral and longitudinal corrugations, shoving, and other movement. The first of these is of primary importance in this report; and, although the others are related, they may be more closely associated with mix design deficiencies.

IMPORTANCE AND RELATIONSHIP TO TOTAL DESIGN SYSTEM

Each of the main topics discussed by Groups A through I can be considered a box within the total framework. Figure 3, as presented by Hudson in Part III, shows that distortion or permanent deformation is one of the limiting distress response outputs of the pavement system. Finn, in his presentation, indicates that, from field observations, the distortion problem should receive second priority after fatigue cracking. Several speakers referred to pavement distress categories such as those referred to by McCullough in Figure 1 of his paper. Within this framework those items considered under permanent deformation are as follows:

1. Excessive loading,
2. Time-dependent deformation (creep),
3. Densification,
4. Consolidation, and
5. Swelling.

Of these, item 2 appeared to be of most concern, whereas items 3, 4, and 5 can be lumped together as volume change. Excessive loading is not the usual condition for design, and, therefore, was not considered further. However, it was felt that items 2 and 3 could be taken as the major causes of traffic-induced permanent deformation.

MATERIALS AND THEIR CHARACTERIZATION

Research and experience have shown the response of most pavement materials to be time-dependent and to be probably affected by the properties of materials themselves, their relationship or proximity to other materials in the system, and the usual factors of load, environment, and so forth. Conventional materials and their suggested form of characterization follow:

1. Asphalt mixtures—linear viscoelastic;
2. Granular bases or subgrades—assumed to be elastic or linear viscoelastic;
3. Cohesive subgrade—linear viscoelastic; and
4. Other materials including portland cement concrete and cement-treated base—assumed to be elastic.

Characterization of these materials in the laboratory for input parameters can best be accomplished by using a triaxial test apparatus. The types of tests deemed suitable would include at least the following:

1. Constant stress or strain (creep),
2. Sinusoidal, and
3. Repeated load.

It is recognized that one or more of these tests may be used to determine both the time-dependent and the volume change responses to loading and environment.

For the purposes of this workshop, it was assumed that other items that may contribute to surface deformation were not to be considered but were to be recognized. For example, construction deficiencies or local design considerations such as the use or lack of soil filters were not a key part of the discussions. However, it was recognized that improvement of various materials such as cement treatment was beneficial in preventing permanent deformation but that, if a systematic approach is used, these benefits will be a natural result of the material characterization and of subsequent analysis within the total framework of the pavement system.

STRUCTURAL ANALYSIS AND PREDICTIVE TECHNIQUES

Essential to the design or analysis process is a method of computing or determining pavement behavior in terms of stress, strain, deflection, or permanent deformation. Following is a brief summary of the status of present methodology.

Existing Techniques

The currently available methods based on stability criteria tend to preclude permanent deformation at least for conventional materials and designs. However, it is suggested that in the present California method, if the resistance value at the subgrade, R, is 10 and the asphalt concrete layer on top varies from 5 to 7 in., no traffic-induced permanent deformation is to be anticipated.

The group was of the opinion that in order to prevent traffic-induced permanent deformation the subgrade needed to be strengthened. To minimize the strain in the subgrade requires that the subgrade have a high bearing value (e.g., R = 10) or be stabilized.

Quasi-Elastic Method

This method developed by Shell suggests that, if the strain at the top of the subgrade does not exceed 6.5×10^{-4}, no permanent deformation could be anticipated for 10^6 repetitions of an 18-kip axle load. The AASHO Interim Guide and the Kentucky method are also based on similar principles.

Linear Viscoelasticity for Layered Systems

So that we can estimate the manner in which deformation accumulates in flexible pavements, a model is needed to account for the manner in which this deformation accumulates as a function of load, environment, and material variables. Specifically, the model should be able to account for the following variables:

1. Time-dependent behavior of materials;
2. Temperature-dependent behavior of materials;
3. Magnitude, duration, and number of repetitions of the loads; and
4. Influence of moisture changes.

A linear viscoelastic model of layered systems that can account for variables 1, 2, and 3 has recently been developed. The model is currently operational and requires that the creep properties of the materials be given in the form of creep compliance functions. It provides the total deflection and the permanent recoverable deformation. The influence of temperature and its variation can be accounted for only if the time-temperature superposition principle is assumed to be valid. The model can now account for randomness of load, temperature, and material properties in a simulative manner by using random number generators.

At the present time it is recognized that there may be a period of testing the technique through various validation procedures. These may include attempts to predict accumulated deformation by using a range of facilities capable of providing the necessary experimental data.

With regard to all three of these approaches, it can be recognized that the first and second are primarily methods of preventing excessive deformation in the form of rutting. However, the quasi-elastic approach has also been used in approximating the amount of rutting to be expected. The third method, based on linear viscoelastic theory, is an attempt to actually permit prediction of accumulated deformation.

MEASUREMENT AND DESIGN CRITERIA

It has been recognized that a measurement technique may be required to identify varying degrees of severity of permanent deformation. These may, in fact, be items such as rut depth, slope, and volume per station. The problem of relating these measurements to overall serviceability or performance is beyond the scope of the subject considered by this group, but it should be one of the long-range research objective. No specific recommendations are offered.

NEEDED RESEARCH

1. Laboratory procedures for characterizing the properties of granular materials used for pavement bases or subgrade or both should be developed. To accurately account for these materials in the pavement structure it will be necessary to characterize their time-dependent properties, if they exist. In addition, it is required that a predictive laboratory technique be developed to account for volume change in the pavement materials, which in turn must be separated from creep or time-dependent behavior.
2. It appears that a potentially valid procedure is available to predict permanent deformation based on linear viscoelastic theory. However, further verification of its validity must be accomplished through actual field measurements or other large-scale representative tests.
3. Once it becomes reasonably feasible to predict rutting or other traffic-induced permanent deformation, it will be necessary to relate these values to performance. It is recommended that a long-range objective be the determination of a deformation measurement system and the role that each degree of deformation plays in the overall performance such as in riding quality.

GROUP H

OTHER PERMANENT DEFORMATION

Chairman, Roger V. LeClerc; recorder, Stuart Williams; members, Richard D. Barksdale, Arthur T. Bergan, J. E. Fitzgerald, C. R. Hanes, Chester McDowell, James M. Rice, Frank H. Scrivner, James F. Shook, Aleksandar S. Vesic, and Harvey E. Wahls

Other permanent deformation, for the purpose of this workshop and this report, was defined as those permanent deformations in the roadway that are not induced by traffic and that do not include cracking, although the distortion might be a precursor to or cause of cracking. The primary cause or source of these deformation manifestations was considered to be hydrothermal volume changes in elements of the pavement structural section and the foundation thereof. Other sources of ancillary concern to the pavement designer, but nevertheless to be recognized, are problems associated with distortion due to differential settlement within embankments or displacement (creep) within embankment foundations.

PROBLEMS AND THEIR PRIORITIES

Discussions relative to definitions and delineation of specific deformation problems soon led to the realization that their import to the pavement designer could not be correctly and properly assessed until means were available for expressing the effects of distortion in terms of pavement serviceability, or lack of it. This then became the ranking problem—the one considered to have first priority in this area: the need to establish a means for measuring the effects of permanent deformation on the roadway in units of serviceability. Without this, the designer has no basis for decision criteria in selection of different designs to accommodate the anticipated volume change. With this, the designer will have a relevance yardstick.

Obviously, the major manifestations of permanent deformation turned out to be those associated with volume change that is nonuniform or differential in nature and is related to expansive soils and effects of freezing temperatures. Some means are now available for identifying and controlling expansive soils, and studies are currently in progress in the United States, Australia, South Africa, Morocco, and Israel. However, more answers are needed for design. Studies on freezing temperature volume changes are also under way, but there is much yet to be learned.

Whereas discussions were thorough on both aspects of volume change, the second most important problem area recommended for study combined these as follows: a need to improve our ability to predict, quantify, and control or design for movements, both total and differential, caused by hydrothermal volume changes in the pavement structure and the underlying foundation.

In the realm of overlay, second-stage, or maintenance design a badly distorted or warped roadway surface may be encountered. This may be the result of volume change activity either fully or partially complete. Such design situations represent the problem area considered next most important: a need to develop economical and effective means for at least semipermanent correction of serious pavement distortion resulting from hydrothermal volume change activity. Current measures are frequently of fleeting effectiveness if the volume change is of a continuing or cyclic nature. Means for permanent correction of the problem are many times quite gross and costly.

Not strictly within the province of the pavement designer, but still something leading to pavement deformation, is the distortion that results from uneven consolidation within an embankment and its foundation and from displacement associated with the latter. Although the pavement designer can do little to control the consequences of this movement by design techniques, he can at least recognize this possibility (or probability) as an element in his decision criteria. Means for predicting and quantifying total settlement are available; no such methods are at hand for differential movement. This then becomes the problem area of fourth ranking: a need for means to predict embankment settlement, both total and differential, due to nonuniform consolidation and creep (flow) of soft foundation soil.

Two other aspects of permanent deformation considered of lesser import or possibly not strictly within the aegis of this group were discussed and are identified here. Additional details on all items are given in the discussion section. Permanent deformation in asphalt concrete from cyclic thermal effects has been noted in Utah Department of Highways laboratory studies but it has not been confirmed by field measurements except possibly deformation manifested by cracking. This information is, therefore, referred to the group concerned with this problem. Possibilities of permanent deformation from subsurface discontinuities (conduits) were noted and deemed inappropriate to our discussion of pavement design. Designers, however, should recognize it and encourage implementation of proper backfill techniques.

DISCUSSION OF PROBLEMS

Group H discussed a number of types of problems considered by various individual members to be pertinent and within the scope of the subject for this group. A brief summarization of the discussion on each item follows.

Volume Change Due to Moisture Variations in Expansive Soils

Although fractures of the pavement surface may result as a secondary manifestation of distress in this category, the group considered this within its subject scope because nonuniform permanent deformation is the principal and most significant result. After considerable discussion, it was the consensus of the group that this is a major problem in a large portion of the country.

An approach to the identification and control of potentially "expansive" soils is in use in Texas and is as known as potential vertical rise determination. In addition, several different methods have been developed elsewhere to alleviate or minimize the overall expansive soil problem. However, these methods have not been proved entirely and consequently are not universally used.

Various group members referred to research activities in progress in Texas, Colorado, South Dakota, Australia, Africa, and Israel. These activities include several fairly comprehensive experimental construction projects in which various methods are under operation.

The consensus was that research is needed to (a) predict and quantify potential movements caused by hydrothermal volume changes, both total and differential, that may occur in the pavement system, and (b) provide an adequate methodology for designing new pavements and for rehabilitating older pavements in areas where expansive soils are a problem.

Volume Change Resulting From Moisture Variations Within the Pavement Structure

Although the problem of volume change resulting from moisture variations within the pavement was recognized by the group, it was not considered to be a high-priority item and was, therefore, omitted from the final selection of the most important problems.

Volume Change of Pavement System Materials Resulting From Freezing Temperatures

Pavement deformations may result from positive or negative volume changes caused by temperatures below 32 F in the pavement system materials. Discussion brought out that the heaving effect that results from the formation of ice lenses has been quite thoroughly studied and that a methodology for solutions to this problem is available. However, it was also brought out that a second type of behavorial phenomenon may occur that results in a decrease in volume with a decrease in temperature below 32 F. This type of deformation is apparently a major problem in Canada and in a number of northern states. The "conventional" expansion of freezing pavement elements or foundations resulting in either uniform or differential displacements was also recognized as a problem.

Adequate solutions to these problems are not available, nor are the problems well understood. Very little related research is in progress. Therefore, recommendations for future research on this item are included as a part of the group's overall recommendation for a study of hydrothermal volume changes.

Creep and Consolidation of Pavement Foundations

A relatively large amount of research on consolidation has been or is being accomplished. However, more work is needed on creep. Also there is a need to obtain information regarding the amount of differential movement that can be tolerated in embankments. Consequently, the group decided to recommend the initiation of the following research studies:

1. Establishment of a means for measuring the effects of permanent deformation on the roadway in units of serviceability; and
2. Development of improved methods for predicting the settlement, particularly differential, of embankments due to creep (flow) of very soft underlying foundations.

As discussions progressed, it became evident that the means for expressing permanent deformation in serviceability terms was a primary need. No means are now available for quantifying the deformation; therefore, evaluation of the extent and severity of the problem is not possible. This item thus became the number one priority.

Other Items

Investigation of volume change (shrinkage) in asphalt concrete due to cyclic thermal changes was reported to be in progress in Utah. Some discussion indicated that this was not confirmed by field measurements but has been possibly tied to transverse cracking. Although the volume change prior to cracking, and the suspected development of internal stresses, was considered within the scope of this group's subject, it was not explored further because the usual manifestation appears to be cracking. Reference of this problem to the appropriate group is made through this report.

A concluding item discussed was that of subsurface discontinuity due to conduit trenches. All agreed that this was a problem but one related only incidentally to pavement design. Consensus of the group was that improper trench backfill techniques would most certainly result in permanent deformation at the pavement surface. However, it was agreed that there was very little the pavement designer could do to rectify poor backfill construction; his efforts would be better spent on endorsing and encouraging proper backfill procedures.

SUMMARY

The deliberations of this group resulted in recommendations for studies or research that, when and if successfully completed, would provide the designer with means for measuring or evaluating consequences of permanent deformation, means for predicting possibility and extent of pavement deformation, means for controlling pavement deformation by design, and means for correcting those situations that have escaped these proper design procedures. However, these research studies will require a considerable length of time because of the complexity of the problem.

Whether the scope and extent of the permanent deformation problems described here will warrant high priority in the overall universe of pavement structural design problems will, of course, have to be evaluated. However, these problems occupy one or two of the "black boxes" in a pavement design system, and the system will not be complete without a solution output.

GROUP I

RELATING DISTRESS TO PAVEMENT PERFORMANCE

Chairman, W. Ronald Hudson; recorder, S. R. Yoder; members, John E. Burke, Fred N. Finn, Wallace J. Liddle, James W. Lyon, Jr., Lionel T. Murray, William H. Perloff, Robert L. Schiffman, Eldon J. Yoder, and Karl S. Pister

Initial discussions centered around a description of the performance function. Performance was generally recognized as the "system output function" or the variable to be optimized in a pavement system analysis. It was generally conceded that performance is defined by some record of the accumulated service of the pavement, i.e., a measure of how well it has served traffic.

MAINTENANCE CONSIDERATIONS

There was some discussion as to when and how pavement failure could be defined. In this respect, the following two questions were asked:

1. When and why are pavements resurfaced?
2. When and why are pavements reconstructed?

These questions led to a general discussion that the group thought would add information to the problem concerning why pavements are maintained. Basically, the following reasons were cited as the ones of major interest:

1. Management decisions;
2. Economic decisions (i.e., funds are available);
3. Desire to prevent deterioration after the observation of primary distress;
4. Poor serviceability conditions that need repair or correction;
5. Safety considerations, such as skid resistance;
6. Stage construction; and
7. Political factors.

Of these reasons, items 1, 3, and 6 seem to be amenable to evaluation by performance determinations. Other reasons seem to be primarily based on management or political decisions beyond the control of the engineer. However, the fact cannot be ignored that a maintenance intervention into the pavement system, for whatever reason, must be considered in subsequent evaluations of that particular pavement.

SERVICEABILITY EVALUATION

Discussions of methods currently in use by various state highway departments were presented at the group meetings. Most of the methods involved a serviceability rating by the road user or an index based on correlation with this rating. The only other major evaluation technique discussed was a mechanistic evaluation. As used by many highway departments, it primarily employs deflection measurements to evaluate the need for preventive maintenance but, of course, not as a direct way of evaluating the mechanism of pavement failure.

By unanimous vote, the group went on record to state that a present serviceability rating or present serviceability index pavement evaluation system is the most satisfactory method currently available for evaluating pavement performance.

Relating Distress to Performance

In considering the factors that affect pavement performance as indicated by the PSI equations from the AASHO Road Test, the group agreed that the effect that has the strongest direct weighting function is roughness. However, it was pointed out that there is a strong correlation between cracking and roughness and that a regression analysis on the pavement serviceability-performance data using cracking and patching as a primary variable (excluding roughness) gives a very high correlation between present serviceability rating and cracking and patching, a correlation coefficient of about 0.8 versus 0.9 for the equation using roughness. This indicates that quite often roughness and cracking and patching occur together in the pavement. The group pointed out that this could be due to one of the following two causes:

1. A pavement cracking up, causing water to enter and, thus, becoming rough;
2. A pavement becoming so rough with heaving or consolidation that it causes the surface to rupture or causes additional dynamic loads, which cause the pavement to crack more rapidly.

The group, in general, thought that this matter needed extensive study.

In summary, it was felt that there was no other way to relate distress to performance, except through some statistical analysis of serviceability-performance and distress information. It was also agreed, however, that any correlation similar to PSI should be carefully formulated and analyzed to determine causation, if possible, and not just correlation. This can perhaps be done by covariance analyses of the various factors involved.

It was pointed out by several group members that limiting deflections or limiting strains, when they are used as design criteria, are basically pseudonyms or stand-ins for a true measure of performance and not failure functions themselves. These can be obtained by observing deflection and subsequent performance as was done on the AASHO Road Test, or they can be evolved by more general experience but perhaps with less accuracy, as with the CBR method.

Quantitative Prediction of Distress

There was a discussion of the ability to quantitatively predict distress. One member pointed out that a true mechanics solution could only predict whether fracture would occur and not the quantity of distress.

What Distress Factors Need To Be Considered?

The distress interface with performance involves deformation almost totally, but some of the deformation is primary; i.e., it results directly from the accumulation of behavioral effects and thus directly affects performance. Other deformation is secondary, such as the deformation that results from water seeping into cracks where cracks are the primary distress factors.

DATA FEEDBACK SYSTEM

The group agreed unanimously that adequate records of feedback information must be kept and added to for evaluation of the design system. The data base or the data system has to be relevant to the problem and contain necessary information; however, data pollution, i.e., the development of unneeded data, is quite often a primary problem in such situations.

The discussions emphasized the importance of cognitive processes in this whole area of work. In general, the computer is a vital tool for handling the large data base involved. Additional study is needed on this subject by highway engineers.

DISTRESS FACTORS

There was a general discussion of how much information about distress factors could be obtained. The group realized that qualitative roughness information can be obtained

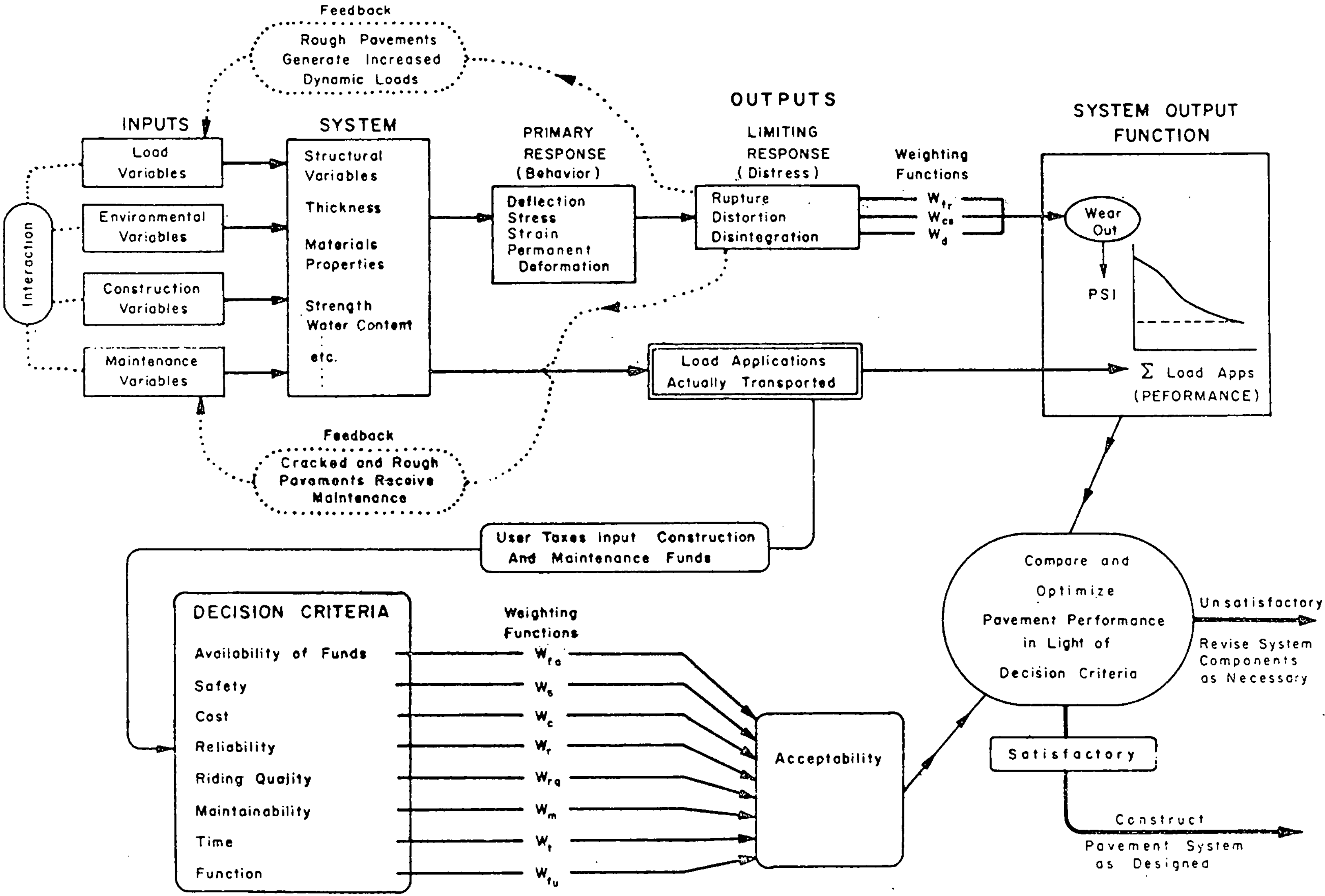

Figure 1. Pavement management system (from Finn et al.).

but that a considerable amount of research is needed to evaluate the effect of wavelength and amplitude of roughness on the vehicle and the driver. In all cases the evaluation of serviceability and performance should consider the driver and the vehicle as part of the roughness system. Additional information is needed on limiting factors that might be involved in performance. An example of this might be rutting where a specific rutting limit might be established as "failure" for reasons of safety. Thus, a pavement with a ½-in. rut might be declared failed. It was emphasized, however, that this condition might be corrected easily with an overlay and that an unsatisfactory or failed condition has nothing to do with the subsequent mechanistic evaluation that would be required to determine how the pavement might function after the overlay. It is this concept and the recycling effect that are necessary to any comprehensive design method.

In further discussions it was pointed out that we must be able to show benefits in order to sell the method and that we in fact need a realistic cost estimate of current methods and future methods to make a realistic benefit-cost analysis of changing the process.

RESEARCH NEEDS AND PRIORITIES

The group identified eight items of research needed to improve the performance function used in pavement design. These items are listed in order of priority in the following.

1. Specify a better relationship among service, performance, time, and traffic. It may be necessary in this context to bring in things such as utility theory (1, 2). The need is for a more definitive relationship that better delineates the factors involved, the weighting functions, and their relationship to the overall process.
2. Develop a more rational relationship between distress variables and serviceability-performance. This may of necessity include stochastic concepts.
3. Initiate a study of maintenance and the effect of maintenance on serviceability-performance trends of the pavement. Although this is considered by some to be an extension of design concept, there may in fact be a stabilization of factors in an existing roadway. This possibility and the problem of field evaluation make the need for research evident.
4. Organize a study to quantify and specify a data system for information feedback, storage, and retrieval. For the entire pavement management process, questions to be answered include: What data are to be obtained? How many data are to be obtained? How are the data to be stored? How are the data to be retrieved? How are the data to be processed and analyzed? Other factors include sampling plans, time, and frames for sampling.
5. Develop and implement such an information feedback system for the pavement management system.
6. Develop a plan for obtaining quantitative distress information.
7. Develop a better relationship between behavior and limiting behavior or distress.
8. Develop cost information on existing design processes and estimates on proposed processes to provide a comparison.

In addition to the eight priority items within the scope of the group's subject, it was felt that there was one priority item of interest to all groups, and this is the development of a systematic program to organize the research and development process in order to develop a rational pavement design system methodology. Such a process must be a total pavement and pavement research management system similar to that shown in Figure 1.

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PART III

STATE OF THE ART

INTRODUCTORY REMARKS

William N. Carey, Jr.

There has been no direct involvement of members of the Highway Research Board staff in pavement design research since the conclusion of the AASHO Road Test reports in the early 1960s. Nevertheless, we have watched with great interest the work done in NCHRP projects at the Ann Arbor conferences, by the Federal Highway Administration, and by many additional researchers to develop a rational pavement design system during the past decade. There have been some real advances in that time, yet the work has been rather disjointed. I hope this workshop brings the ideas together and establishes a basis for coalescence and for a more controlled and directed attack.

It is appropriate to look back at the big road tests of the 1950s. We did what we set out to do. The main question in those days was, What is the relative effect of different axle loads and numbers of load applications on pavement performance? We found first that until then there had been no definition of pavement performance, so we formed a definition from the concept of serviceability as it deteriorated with time and traffic. There has been a good bit of picayune criticism of some of the details, but the concept has held up until now.

The answer to our major question required controlled-traffic experiments in which distress could be associated with specific known traffic. Controlled tests are expensive; the road tests cost a lot. However, we did learn the relationships between traffic and load and pavement performance to a degree of precision and usefulness such that we do not feel other controlled traffic tests are needed until we know much more about other variables and wish to move to a new degree of refinement.

We had hoped that a nationwide experiment involving road test satellite studies would be undertaken. We argued long and hard for such an experiment. We were opposed by sincere men who felt that universal pavement design equations developed from classical mechanics were just a few years away, so there was no point in spending money on further nonmechanistic types of road test approaches. I, for one, still believe that, had we had the satellite studies, by now we would be on the way to a useful universal design system. Instead, I still hear clever doctoral dissertations on how to predict stress in an element of a homogeneous elastic slab on an idealized foundation. No one has ever suggested how knowledge of such stress relates to the performance of pavements in the real world.

Nevertheless, I am told that now advances have been made and that this workshop will serve to bring together the knowledge that has been accumulating among researchers and designers during the past few years. I am fully confident that this is true and that this workshop will provide strong American input to the Third International Conference on Structural Design of Asphalt Pavements in London in 1972. I hope that this knowledge will include evidence of the movement from theory to practice that is indicated on our program.

I do hope we are getting someplace. Everyone who is honest knows that we have been designing pavements by black magic for 40 years. This was acceptable when traffic was light and when anything the highway departments did was a step forward and was welcomed with shouts of glee from the public. As you know, traffic is no longer light, and the public is at least confused about its highway programs. The Highway Research Board-NCHRP public opinion survey showed that the public loves its automobiles but is quite indifferent about highways. In fact, there is a good bit of evidence that the public is negative about highways these days.

So, when the Government Accounting Office starts criticizing our pavement designs (which they have and with embarrassing justification), we had better hurry to get some rational answers. When our Interstate highways show serious distress in 5 years (which they have), we need a better defense than, "The contractor did not build it right," or "The soil at that spot was not what it was supposed to be," or "There are more trucks than we guessed there would be."

I believe the people at this workshop hold the key to the answers to these questions. I believe the people at this workshop are more important to the future viability of highway transportation as an acceptable transportation mode than they realize. Of course, we know that there are relatively few pavement failures in terms of mileage built, but there are too many, and 100 feet of major repair can close, or at least confuse operation on, a 10-mile stretch of nice new Interstate. Sooner or later the public will refuse to accept apologetic excuses for design failures. This workshop must lead the way. It is no longer a minor skirmish—an interesting intellectual exercise; it is a serious situation for all of us and for American transportation.

SOME REMARKS ON RESEARCH FOR STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENT SYSTEMS

Karl S. Pister

That construction and operation of successful pavements predates the dream of a "rational" method of design can hardly have escaped the attention of engineers or laymen. The present state of the art is the product of a long history of successes and failures, the former fortunately overshadowing the latter. In fact one may well ask the question: Why does the engineer want a "theory" of pavement design, inasmuch as theories invariably are wrong, have limited applicability, or are too complicated when put to the test of real experience? The answer seems to lie in the observation that, in the present milieu of rapid change, experience quickly becomes obsolete or is often totally lacking. Examples of this are rampant in the pavement field; e.g., witness the increase in traffic volume, the change in construction costs and methods, and the potential problems arising from disappearance of high-quality raw materials with concomitant increased use of new and sometimes marginal substitutes (would you believe crushed glass bottles?). The Via Appia was a first-class Roman road; but, as any tourist can tell you, the service life has long been exceeded. Thus, it appears that this workshop was based on the implicit assumptions that a rational method of pavement design exists, is important to acquire, and is accessible to the minds of engineers. In reviewing the papers prepared for the workshop, I have drawn the conclusion that these assumptions are shared by the speakers, and I expect they are held plausible by most of the participants. However, as we are often painfully aware, sharing a common set of assumptions does not imply any uniqueness for subsequent application and action. This I believe is what needs very careful examination during the workshop sessions. We must cast aside our denominational prejudices and try to examine what indeed we are trying to accomplish from our common point of departure. We will then be in a much more favorable position to discuss the organization of research and development work directed toward sucessful design and management of pavement systems. In this regard the following quotation from Bertrand Russell's "Unpopular Essays" is quite relevant: "So whenever you find yourself getting angry about a difference of opinion, be on your guard; you will probably find, on examination, that your belief is getting beyond what the evidence warrants."

Let me now return to the "implicit assumptions," which in a sense form the basis for my subsequent remarks. It seems to me that the existence of a rational method of design has to be established a posteriori; i.e., it is the task of the engineer to observe, acquire, and organize information and experience obtained from operational physical systems. This point of view, incidentally, is strongly supported in Finn's paper (9). However, these steps are in themselves insufficient, for we must perform the extremely difficult job of "pattern recognition" to visualize the structure of a model that, post facto, seems to "fit" our observations. Such a model, given an adequate mathematical structure, can then be employed to carry out simulation tasks to seek the "best" among alternatives—this being the pragmatic task to which we customarily attribute economic importance, whether in terms of dollars or expenditure of other resources. In turn, this step leads to the development of the final assumption, accessibility of the rational method to the engineer. The kinds of models and methods to be discussed here are from a practical viewpoint accessible only through a digital computer, arising from experience with operating pavement systems.

Finally, before we turn to a more systematic examination of the ideas sketched earlier, it is well to note that, even though we accept the existence of a rational design formula, we do not pretend to believe that, like the commandments of Moses, the formula is unchangeable with time. Man (especialy engineers) by nature is a creature who likes to modify his systems, adapting them to suit his needs. Pavement design and management are problems of adaptive control, a concept that is found to be extremely useful in development of models of system design and behavior. We shall, in fact, try to indicate how insight into design and management of pavement systems may be enhanced by viewing the task as a multistage decision process, utilizing Bellman's dynamic programming techniques as a vehicle. Let us turn now to certain preliminaries and terminology needed to describe the problem under consideration.

A FLEXIBLE PAVEMENT SYSTEM

As a point of departure we adopt the terminology used in HRB Special Report 113 (1).

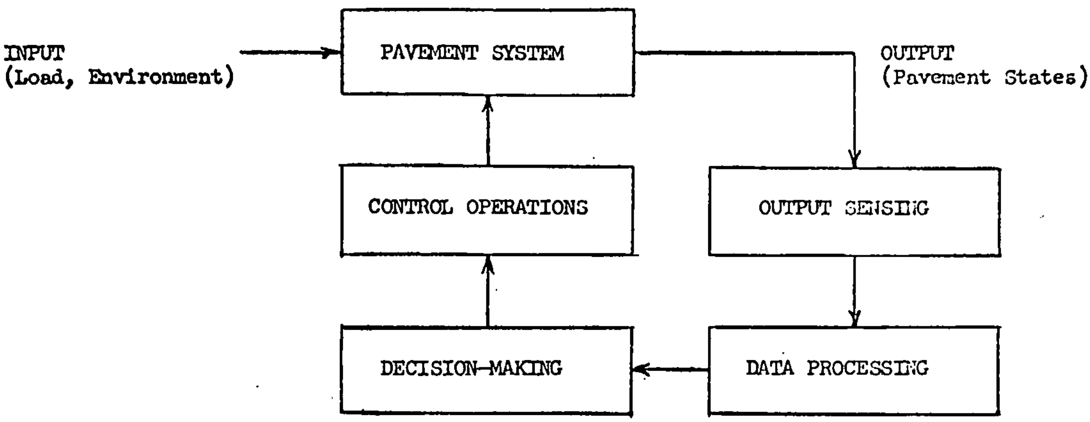
1. A flexible pavement is a pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.
2. Pavement structure is the combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed. To this I would add the qualifying phrase, "under a history of environmental conditions."
3. Serviceability, which embodies the function of a pavement, is the ability of a pavement to serve traffic with safety and comfort and with a minimum of detrimental effects to either vehicle or pavement.
4. The present serviceability index, which is the current (present) measure of the effectiveness of the pavement, is a numerical index of the ability of a pavement in its present condition to serve traffic.
5. Performance is the measure of the accumulated service provided by a pavement, i.e., the adequacy with which a pavement fulfills its purpose. I would add here that performance implicitly includes "service per dollar" or some other type of economic measure.

These definitions taken together constitute what we may call a flexible pavement system: a set of interacting components subject to various inputs (traffic, environment), producing various outputs (as yet unspecified). System performance measures adequacy over the operational lifetime. At this stage there is little to be gained from a more formal definition; however, there is a great deal of conceptual mileage to be gained from this intuitive picture. For example, it makes clear that performance is the real goal of design and operation (through proper management) of the system. Yet, relatively little information concerning pavement behavior, in which performance is the dependent variable, can be found in the literature. This is of course understandable because performance is somewhat ambiguous to define, in spite of its conceptual importance.

Perhaps it is easier to deal with distress, which is really absence of serviceability, and to invent measures of distress, along with a normalizing requirement that "serviceability plus distress equals unity" during the pavement lifetime. No matter what definition prevails, the point here is that one must acquire sets of "systematic and continuous observations of performance (or distress) of full-scale pavements" (9). It is only through such a data acquisition program that any hope of pattern recognition will emerge to guide the formalization of operational rules leading to rational design. For example, without this, mathematical simulation of pavement systems, no matter how fascinating a game in itself, will remain precisely a game with very little payoff to pavement systems. Whether one adopts distress (a structural or mechanistic, designer-oriented concept) or performance (a user-oriented concept), as a practical matter it is expedient to attempt to divide traffic-associated performance from environment-associated performance wherever possible. In this connection frequency of occurrence studies of types of distresses serve to emphasize the behavioral aspects of the pavement system deserving the most detailed study. Such observations necessarily are global in nature; i.e., they constitute integrated or averaged values of pavement response variables, as opposed to local values of the variables. This is in fact an extremely important point that strongly influences the development of mathematical models, as will be seen.

Finally, it may be appropriate here to consider the advantages of dealing with the pavement system problem in two separate, yet highly related, stages: (a) the problem of observing and controlling (managing) an existing pavement system to achieve optimum performance, and (b) the problem of simulating a pavement system by mathematical modeling so that an optimum design configuration can be achieved. These problems cannot really be separated either in planning a research program such as is our task here or in implementing a policy for design and control. As noted previously, a separation of these problems leads to the probability of two separate games being played rather than one; thus caution must be observed. When these problems are examined, it is useful from both conceptual and operation viewpoints to use block diagrams to describe the system under consideration. Figure 1 shows the basic elements of a system whose output is analyzed and evaluated by performance criteria so that control operations (maintenance) can be effected to provide a certain level of serviceability. No attempt is made at this point to inquire in detail into the subsystem components constituting the system, nor to select quantitative measures to describe the system. Obviously, this is a crucial matter for the success of mathematical modeling, and it will be examined more fully later in this paper.

A similar diagram can be constructed for the second problem of mathematical simulation of a pavement system. The emphasis in this problem is on selection of the parameters of the system itself; i.e., for a given range of inputs and desired performance criteria, a policy leading to an optimum selection of model parameters is desired. This is the classical inverse problem of design, a problem whose complexity invariably requires that a certain family of model structures be examined, from which the "best" choice of parameters is selected. (An example is the selection of layer thicknesses and elastic moduli using elastic layered system theory as the mathematical model.) A diagram of the basic phases of mathematical simulation is shown in Figure 2.



INPUT

Traffic-associated
Environment-associated

OUTPUT SENSING

Visual inspection
Transducer measurements
Test vehicles

DECISION-MAKING

Select optimal policy

OUTPUT

Stress
Deformation Performance
Distress

DATA PROCESSING

Evaluate performance quantitatively

CONTROL OPERATIONS

Maintenance
Repair
Replacement

Figure 1. Pavement system control.

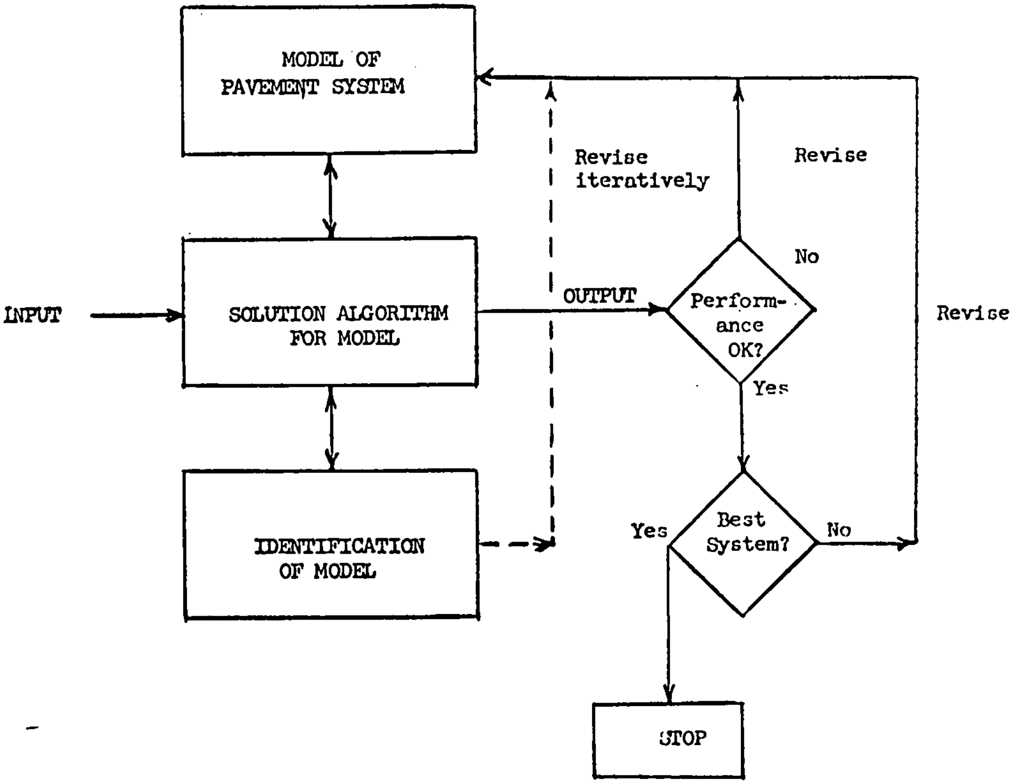


Figure 2. Simulation of pavement system.

A treatment of the design and management of pavement systems as a control problem can be found in the paper by Hudson (10), as well as in an earlier report by Hudson et al. (4). These papers call attention to the need to view the problem in the context just described. In addition, however, they suggest a structure by which quantitative results can be developed. Because of the extreme complexity of the system, the method is necessarily rather primitive (even if somewhat involved). Nevertheless, it should be considered an important step forward. What I wish to emphasize in referring to this work here is the need to develop a general systems model not only describing the broad problem of design and management but also looking critically into the "black box" so that each subsystem is understood and described in the most comprehensive way possible. This workshop provides the opportunity of bringing together people who look at various "black boxes," and it is incumbent on us to bind ideas together into a cohesive view of the real problem. Short of this we will return to playing our own games of solitaire.

The systems viewpoint described in this section is drawn from ideas presented in a report by Hudson et al. (2), which in part grew out of an earlier version of pavement systems analysis (3). Since that report, substantial progress in applications has been made, notably by Hudson and his colleagues (4) and in the very interesting works of Lemer and Moavenzadeh (5) and Moavenzadeh (11). What seems most appropriate to the writer at this time is to exploit to the fullest the structures and methodologies currently existing in the field of "systems." I use this term advisedly, because there is often a certain hesitation or snickering among engineers at mention of the word. Let me make clear my intentions. First of all, the notion of a system has been helpful to organize and place into proper interrelationship the myriad factors influencing behavior of a pavement structure. Furthermore, as more is learned about actual pavement performance through systematic field observation, systems engineering provides the fabric on which an interaction matrix can be constructed, i.e., the possibility of assigning weighting factors to system subcomponents so that more enlightened research and development can be undertaken in areas with highest payoff. This phase of the application of systems theory can be qualitative or semiquantitative (e.g., the insights gained by examining block diagrams, categorizing the type of distress, and the like) and still be of considerable value in guiding design practice and in orienting research and development.

The second aspect of systems, one to which I wish to devote attention later in the paper, is that commonly associated with system control processes and dynamic programming. The notion of pavement design as a feedback control process is given in the paper by Hudson (10), while Nair (12) points to the probable desirability of examining applications of dynamic programming. With this encouragement I believe it worthwhile to examine briefly the mathematical theory of control to see what light may be shed on our design problem. The formulation of a workable model will require the utmost support from each of the areas represented in this workshop. However, I hope to avoid the mistake of adding a new game to the plethora already at our disposal for entertainment of highway engineers and researchers. Let me first review in more detail the modeling of pavement system simulation (the second problem), which in turn leads logically to the first problem of management through control.

MATHEMATICAL MODELING OF THE DESIGN PROCESS FOR A PAVEMENT SYSTEM

The essential ingredients of design by mathematical simulation are as follows:

1. A description of the configuration and the input-output relations of the system along with a "parameterized" structure defining these quantitatively;
2. A statement of the operating conditions (input);
3. An algorithm for predicting the evolution of the system, i.e., its output or performance;
4. A criterion function by which performance can be judged; and
5. Modification of the system to seek an optimum performance.

The task of the designer is that of searching to select values of parameters of the system (within logical constraints), which in turn lead to optimum performance as judged by the criterion function. However appealing this view of design may be conceptually, it implicitly contains the seeds of its own destruction. Except for the most trivial cases it cannot actually be made operational at this time because of our lack of understanding of realistic input-output relations for the system (13, 14, 15), the difficulty in finding a prediction algorithm (12), and the problem of defining a criterion function (10, 11, 16). These are the subsystem black-box problems to which I previously referred. They deserve our careful attention. In the meantime we have to be content with the best of a bad situation, but we should be careful that our modeling is done by appropriate principles.

Historically the principles used to develop models of pavement behavior have come from continuum mechanics, particularly mechanics of solids, and there does not appear to be any serious challenger at this moment. Let us review briefly how reality appears to a solid mechanician. First, certain state variables must be introduced for the system, in our case, the stress matrix $\underset{\sim}{\sigma}$, the strain matrix $\underset{\sim}{\epsilon}$, temperature T, and possibly moisture content M. Because stress and strain each require six components for their description, we have 14 local state variables, i.e., 14 scalar functions of time at each and every particle (point) in the pavement system. These variables must satisfy certain basic principles of mechanics, such as balance of momentum, conservation of mass, balance of energy, and the entropy production inequality. In addition, when a process is defined for a particular kind of material, a constitutive equation must be identified. The kinds of processes of interest to us here are primarily mechanical—e.g., deformations—although temperature and moisture content may also change. Constitutive equations tell us how these state variables are related during such processes. Different kinds of materials have different kinds of constitutive equations for the same process. The task of "material characterization" is that of determining the nature of the constitutive equations for materials of interest in pavement design. Unfortunately, the processes for which these equations must be found are not known a priori. Thus, the problem of characterization must be attacked iteratively; i.e., assumed equations are used to predict the output (process) of the system so that these processes can in turn be used for characterization experiments, from which data assumed versus actual behavior can be adjusted iteratively, by changing the constitutive model, to a desired degree of accuracy.

Two points need further amplification here: What guidelines are there for selecting constitutive models, and how can the system output be predicted? These questions are discussed in more detail by Westmann (13) and Nair (12), so I will include only a brief statement. Constitutive equations are expected to satisfy a fundamental principle of determinism; i.e., the past determines the future. In solid mechanics this means that the (local) stress state at the present time may depend on all the past states up to the present (history) of strain, temperature, and moisture content. Symbolically, this can be written

$$\underset{\sim}{\sigma}(\underset{\sim}{x}, t) = \underset{s=-\infty}{\overset{s=t}{\underset{\sim}{F}_1}} [\underset{\sim}{\varepsilon}(\underset{\sim}{x}, s), T(\underset{\sim}{x}, s), M(\underset{\sim}{x}, s); \underset{\sim}{x}, t] \qquad (1)$$

Without going into many detailed points that may be raised (see any modern book on solid mechanics), we can say that we have here a statement that the stress matrix at a particle, $\underset{\sim}{x}$, and the current time, t, depends on all past strain states, temperatures, and moisture contents at the particle. The constitutive rule (functional), $\underset{\sim}{F}_1$, may depend on the particle, $\underset{\sim}{x}$, and on time, t; this is clearly so in a layered system and in cases where asphalt properties degrade with time. Generally, it is assumed that, while stress depends on M and T (moisture content and temperature), they can be determined separately from diffusion equations unaffected by input fluctuations of stress. The literature is replete with work reported to have completed the task of finding $\underset{\sim}{F}_1$; however, the facts do not support this contention. One cannot verify most of the work simply because the actual process (sequences of states) in a pavement system is unknown. What one can measure is only a set of selected output variables such as surface deflection under wheel loads, an output known to be notoriously insensitive to constitutive model parameters.

I do not wish to deprecate serious attempts to understand and model constitutive behavior. These are urgently needed to provide the comprehensive subsystem support to which I referred earlier. I wish only to caution against the overenthusiastic approach often used by those seeking support for games and to emphasize that it makes no sense to use additively a set of measurements taken in part with a micrometer and in part with a yardstick. Our resources will be better used if we try to measure the entire problem with the yardstick first and then try to determine the size of the components more accurately, to speak analogically.

Troubles do not vanish with the constitutive problem. We must next confront the task of developing an algorithm for predicting the evolution of the states of the system. In mechanics this is accomplished through the device of an initial-boundary value problem. The term initial suggests that the evolution of the pavement state variables will depend on a starting point in time, whereas boundary suggests that the geometrical confines of the system are acted on, in our case by traffic loads and environment. The solution of such a problem depends on mathematical analysis—primarily numerical analysis performed on a digital computer. The output of the computer corresponds to the output of the pavement system in the sense that states of the system are determined from the input as functions of time and location in the system. Unfortunately this information by itself is inadequate; one must append a criterion function by which output can be judged good or bad. This raises very serious questions that are addressed in the papers by Moavenzadeh (11) and McCullough (16): What is a suitable criterion function? How does one define "failure" locally? How does failure (or distress) propagate in space and time? When does an accumulation of local failures constitute global failure (distress) in the system? These questions deserve a great deal more attention than they have received, and it will only be through serious cooperative efforts of theoreticians and field engineers that any hope of solution will emerge. In symbolic terms one can pose the question: How can the local distress be calculated as a function of time? A possible form is

$$D(\underset{\sim}{x}, t) = \underset{s=-\infty}{\overset{s=t}{F_2}} [\underset{\sim}{\sigma}(\underset{\sim}{x}, s), \underset{\sim}{\epsilon}(\underset{\sim}{x}, s), T(\underset{\sim}{x}, s), M(\underset{\sim}{x}, s)] \tag{2}$$

The scalar-valued functional F_2 at each particle $\underset{\sim}{x}$ assigns a value, D, at time, t, to the set of stress, strain, temperature, and moisture content histories. That number is called here the distress. The structure of the functional F_2 is unclear; some structures appear in the paper by Moavenzadeh (11). The second important question relates to the the propagation of distress from particle to particle, leading to global and often observable damage in the pavement. This notion is embodied in the definition of a second functional defined now over all particles in the system at time t:

$$D_s(t) = F_3[D(\underset{\sim}{x}, t)] \quad \text{over all } \underset{\sim}{x} \epsilon S \tag{3}$$

This formalism calls attention to the idea that $D_S(t)$, the distress in ths system at time t, depends on the accumulation of histories of distress at all particles in the system; i.e., it is some kind of spatial influence function. D(x, t) is related to the notion of (local) distress index defined by Hudson et al. (2), whereas Eq. 3 is a measure of present distress, or perhaps of a volume density of distress, in the system as a whole. In this sense it is complementary to the definition of present serviceability index referred to earlier, and one could write

$$D_s(t) + PSI(t) = 1 \tag{4}$$

if a suitable normalization is performed. System performance, PF, an integrated (over time) concept suggests the definition

$$PF(t) = \int_0^t PSI(s)\, ds = \int_0^t [1 - D_s(s)]\, ds \tag{5}$$

This view, even if it could be implemented through appropriate mathematical structures, still has the disadvantage that it is mechanistically oriented; i.e., the user is excluded from influencing the evaluation of performance. More sophisticated qualitative incorporation of the user is mentioned in other papers (4, 5). These aspects of performance need much more attention than they have received. At the moment it would appear that very limited progress has been made in quantifying the concepts described by Eqs. 2 and 3. One could mention several examples:

1. For linear elastic and viscoelastic layered system models, calculation of maximum surface deflections under simple wheel load patterns is an example of a "functional defined over all particles." If this deflection is limited by an inequality, a crude distress model corresponding to Eq. 3 is obtained. Surface curvature can be treated similarly.
2. For certain types of linear viscoelastic layered system models, permanent surface deflections can be calculated. These would represent a model similar to Eq. 3, in which history is incorporated.

When one moves beyond linear elastic and linear viscoelastic models, a substantial amount of empiricism is introduced. Although there is no evil in empiricism, it should not be listed under the rubric of mechanics. It is a useful procedure that one must employ to obtain a solution of a real system problem in the face of complexity. This leads to the next consideration, that of introducing an element of "external disturbance" into the design process. What has been done to date seems to fall into the pattern of using rational, yet inadequate, models of pavement behavior, observing that these simulations do not correspond to real system behavior and that no rational criteria for distress (or performance) exist and then making the best of a bad situation, namely, allowing the engineer to use his judgment to assign criteria required to achieve as near an optimum as possible for the system design. In other words, the engineer is a short-circuit of the rational design process. Our attempts should be directed toward using the engineer in this role but supplying him with the best possible data on which to base his judgments, thereby minimizing the possibility of irrational short-circuits. The engineer is a finite, fallible control system. In spite of improvements in modeling subsystems and in developing more sophisticated models of the pavement system, predicted performance will seldom match actual performance of a pavement system. In other words, the real system performance leaves something to be desired. This is precisely what constitutes the notion of control in a decision process. Because we do not like the manner in which the system is evolving, we intervene to change its performance. Such control may take the form of patching, seal coats, overlays, or, in an extreme case, complete replacement. We shall now try to sketch more abstractly the mathematical structure of this type of system and indicate a possible direction of research in this area.

PAVEMENT MANAGEMENT AS A MULTISTAGE DECISION PROCESS

We have examined the manner in which mathematical simulation can be carried out in the areas shown in Figure 2. In this section we turn our attention to the problem of observing and controlling an existing pavement system to attain optimum performance along the lines shown in Figure 1. Much of what has been said already pertains to this problem; however, it is necessary to adopt a more modest view of measure of performance in order to expect numerical results, a fact already noted in connection with work reported by Hudson et al. (4). The complexity of the system with which we are dealing suggests the desirability of beginning with a qualitative discussion of the problem and proceeding to incorporate more factors into the model in order to approach a more realistic simulation of the pavement system.

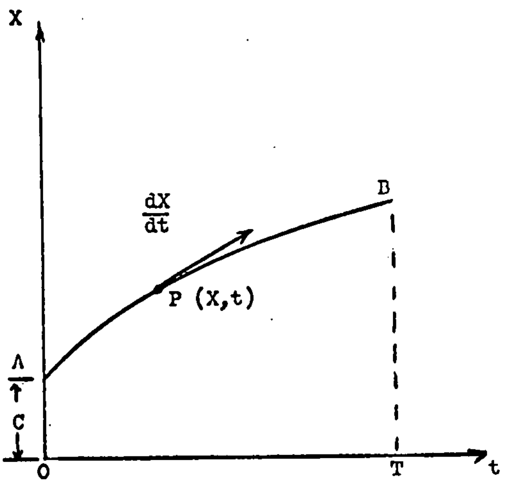
The basic idea in treating the control of a dynamic system as a multistage decision process (6) can be most easily grasped geometrically. Suppose that X(t) denotes the position vector of a particle moving along a space curve. The trajectory of the particle is to be determined in such a manner that the "cost" of moving the particle over its trajectory is minimized. Intuitively, such a process can be thought of as a guidance (control) process in which continuous "steering" directions are required. Thus, a multistage decision process is defined to consist of the following operations:

1. Observing the system state X(t) at time t;
2. Processing this information and making a decision utilizing a control rule; and
3. Modifying the evolution of the system by exerting the control. In the example chosen, at a point P(X, t) along the particle trajectory we wish to determine dX/dt, i.e., the "steering direction" as a function of position (state) and time, such that the cost of the trip is minimized. In symbolic form we can pose this problem by seeking a function G(X, t) such that

$$\frac{dX}{dt} = G(X, t), \text{ with the initial condition } X(0) = C \tag{6}$$

along with

$$K[X(t)] = \text{minimum}, \; 0 \leq t \leq T \tag{7}$$



Equation of Evolution

$$\frac{dX}{dt} = G(X,t)$$

Initial Condition

$$X(0) = C$$

Criterion Function

$$K\left[X(t)\right] = \text{Min.}\ , \quad 0 \leq t \leq T$$

Figure 3. Trajectory controlled for minimum cost.

The problem, as shown in Figure 3, can also be posed in a slightly different manner to emphasize the aspect of control, leading to an algorithm of dynamic programming. Figure 3 shows that the direction dX/dt represents the control variable, which we define as

$$Y(t) \equiv \frac{dX}{dt} \tag{8}$$

Figure 3 also shows that the cost K will depend on the initial state C and the "life" of the trajectory, T. Therefore, we replace Eq. 7 with an equivalent statement

$$F(C, T) = \min_{Y(t)} K[X(t)] \tag{9}$$

In words, we seek a set of "controls," $Y(t)$, which minimizes the cost of a trajectory, parameterized by the initial state and life (duration) of the process. [As pointed out by Bellman (6), this is equivalent to defining a geodesic in the trajectory space in terms of its tangents.] Dynamic programming provides the computational algorithm for determining the set of controls for the process. This set constitutes an optimal policy for guidance of the process, which must satisfy the requirement that the criterion function K attain a minimum value.

In applications to pavement systems two things are apparent: (a) the state and control vectors, as well as the criterion function, are extremely complex; and (b) observations and decisions are made at a finite number of times, i.e., the process is discrete. This leads us to consider a model of discrete deterministic multistage decision processes, in which we now consider a sequence of states $X_1, X_2, \ldots, X_N$ and a sequence of control vectors (decisions) $Y_1, Y_2, \ldots, Y_N$. We define the evolution of the N-stage process by the equation

$$X_n = G(X_{n-1}, Y_{n-1}), \; n = 2, 3, \ldots, N \tag{10}$$

The meaning of Eq. 10 is that, at the initial state X_1 of the process, a decision Y_1 is made. This results in a new state X_2 given by Eq. 10:

$$X_2 = G(X_1, Y_1) \tag{11}$$

and so on, each new state depending on the immediately previous state and decision. Equation 10 corresponds to Eq. 6 in the continuous case. The set of decisions must be chosen such that the criterion function K is minimized, i.e., if

$$K = K(X_1, X_2, \ldots, X_n; Y_1, Y_2, \ldots, Y_N) \tag{12}$$

the purpose of the decision process is to choose the Y_n so as to minimize Eq. 12. At this point the structure of the transformation function G is unspecified, except that it must lead to a unique state. (Although X_n in Eq. 10 depends only on the previous state, it is possible to extend the structure to incorporate hereditary effects, at the expense of computational complexity.) It may depend on the age of the process. The criterion function is presumed to possess a so-called Markovian property so that after k decision, the effect of the remaining (N - k) decisions on the value of K depends only on the system state at time k and the subsequent decision. An additive cost function K satisfies this requirement, i.e.,

$$K = f(X_1, Y_1) + f(X_2, Y_2) + \dots + f(X_N, Y_N) \tag{13}$$

Principle of Optimality

We now consider how to establish the optimum set of decisions (optimal policy) for the problem posed by using dynamic programming. Bellman (7) states: "An optimal policy has the property that whatever the initial state and the initial decision are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision." The proof of this intuitive concept is virtually self-evident. Figure 3 shows that, if AB constitutes an optimal path, having arrived at P, PB must also constitute an optimal path. By using this principle we can deduce a recurrence equation for constructing an optimal policy, given the transformation function G, criterion function K, and the initial state of the system. Given (X_1, Y_1) the system is transformed to state X_2 according to Eq. 10; i.e., $X_2 = G(X_1, Y_1)$ and the N-stage process is reduced to an (N - 1)- stage process. Analogous to Eq. 9, Eq. 14 has the minimizing condition embedded in the initial state X_1 and "duration of process," N:

$$F_N(X_1) = \overset{\min}{Y_n} (K) = \overset{\min}{Y_n} [f(X_1+Y_1) + \dots + f(X_N, Y_N)] \tag{14}$$

From the principle of optimality, and the Markovian structure of K, the cost of the last (N - 1) stages after making the first decision Y_1 will be

$$F_{N-1}(X_2) = F_{N-1} [G(X_1, Y_1)] \tag{15}$$

Thus, it follows that

$$F_N(X_1) = f(X_1, Y_1) + F_{N-1} [G(X_1, Y_1)] \tag{16}$$

From Eq. 14 this choice of Y_1 must be such that the right side of Eq. 16 is minimized. Thus,

$$F_N(X_1) = \overset{\min}{Y_1} \{f(X_1, Y_1) + F_{N-1} [G(X_1, Y_1)]\} \tag{17}$$

Allowing N to range over the values 2, 3, ... produces a recurrence relation connecting members of the sequence $[F_N(X_1)]$, thus specifying an optimal policy. We note the important result: The problem of selecting N decision vectors Y_n in an N-dimensional policy space is reduced to sequential selection of N vectors in a one-dimensional space. The computational significance of this result is obvious. One may well ask why dynamic programming should be selected over a straightforward search procedure that explores all possible policies and selects the policy leading to minimum cost. The answer is that the principle of optimality limits the choice of policies to those in the neighborhood of the policy for a minimum of the criterion function. Policies of no importance are thereby eliminated, along with attendant savings in computational time, a factor of prime importance in multidimensional state vector problems.

We turn now to a simple example chosen to illustrate application of the dynamic programming algorithm (Eq. 17) and associated concepts.

Example: An Elementary Model of Management as a Multistage Decision Process

Let us suppose (contrary to the consensus of speakers at this workshop) that performance serviceability index can be measured and is in fact the sole performance state variable X. Furthermore, it is supposed that the state variable is observed over the life of the pavement at some specified number of times. If no control over the system is exercised, a history of traffic and environmental inputs will cause a monotonic decrease in PSI, which is symbolically shown by Eq. 10 with $Y_n \equiv 0$,

$$X_n = G(X_{n-1}),\ n = 2, 3, \ldots, N \tag{18}$$

and is also shown in Figure 4, labeled 0 to denote zero cost of control. The initial state of the system X_1, normalized to unity, and the structure of the transformation G between states clearly depend on the initial design of the system. Furthermore, the transformation G also depends on the load and environmental inputs carried between state observations. Such a function clearly has to be born of field and road test experience. In order to "manage" our model system let us now introduce the notion of control via Eq. 10, where the set of decisions, Y_n, constitutes alternative maintenance, repair, or replacement operations. We seek an optimal policy for selecting these decisions in the face of certain restrictions, which are in part arbitrary but essential. Here, for simplicity, we choose as our criterion function minimum cost (Eq. 13). In Eq. 13, K represents the accumulated cost of performing the Y_N control operations. (The added effect of "cost of money" can also be included here.) As an added constraint to the

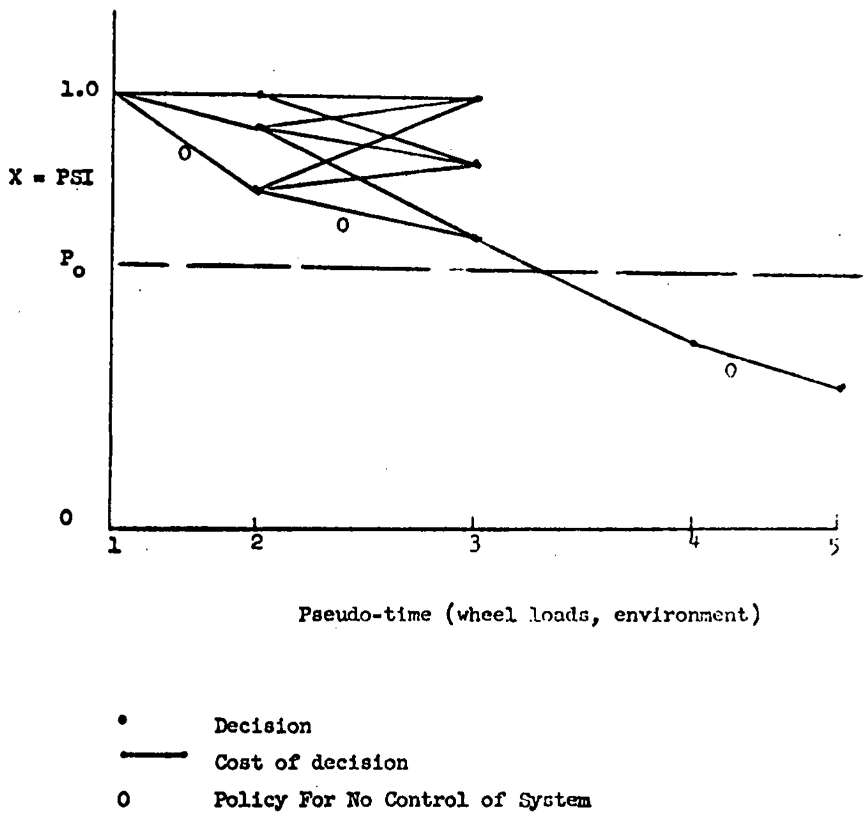


Figure 4. Routing graph of system performance model.

problem, we stipulate that the performance of the system be such that the mean value of the PSI exceeds some minimum value P_O, i.e.,

$$\frac{1}{T} \sum_{n=1}^{N} X_n(\Delta t_n) \geq P_o \qquad (19)$$

where again T denotes system life and Δt_n denotes an interval between states, both measured in some pseudo-time. We must now select a sequence of decisions $\{Y_n\}$; i.e., find an optimal policy such that Eq. 13 is minimized subject to the constraint (Eq. 19). This is a straightforward problem in dynamic programming using the algorithm shown in Eq. 17 modified by a Lagrange multiplier to handle the constraint (7). One forms the modified function obtained by combining the criterion function K in Eq. 14 with the constraint condition in Eq. 19, using an undetermined Lagrange multiplier, λ.

$$F_N(X_1) = \min_{Y_n} \left\{ \left[f(X_1, Y_1) + \ldots + f\left(X_N, Y_N\right) \right] - \lambda \left[\frac{1}{T} \sum_{n=1}^{N} X_n\left(\Delta t_n\right) - P_o \right] \right\} \qquad (20)$$

when a value is chosen for λ, the dynamic programming algorithm (Eq. 17) is used to obtain a policy for the decision sequence Y_n. After this the inequality (Eq. 19) is checked. The process is then repeated by selecting values of λ and repeating the same process until a policy is found for which Eq. 19 is best satisfied; this constitutes the optimal policy. The parameter λ is an important index of price of the control process; in this case it shows the trade-off in cost per unit of performance required to maintain a certain average value of PSI during the pavement life. Other types of constraints can be treated in a similar fashion. In such problems, certain concepts associated with graph theory can be helpful (8). One can view the choice of decisions as a routing problem. At each state, a set of points denotes new states produced by decisions (Fig. 4). The paths from state to state can be associated with costs of control, and one must select the path of optimal control, bearing in mind that the minimum performance criterion has to be satisfied for the set of decisions.

The model considered is clearly an elementary one, but it can be considerably embellished. When the mechanics of management of systems are better understood, the initial design (inverse) problem might be incorporated as part of the decision process. In this instance the optimal policy is to be found over a set of parameters describing control variables as well as design parameters of the system itself. An example of this type can be found in the report by Hudson et al. (4).

Uncertainty

Thus far we have made the tacit assumption that all aspects of the systems with which we are working are deterministic. This applies equally to input, system model, and control. In other words we are certain of the input, which in turn leads to a certain output, and a control applied to the system produces a certain change of state. Use of the term certain is equivalent to assigning a probability of unity in each of these instances. It is a euphemism to assert that pavement system analysis and design is an uncertain problem. Aspects of this overall problem are discussed by Sherman (15) and Moavenzadeh (11), and a suggested treatment of the overall systems problem is mentioned by Lemer and Moavenzadeh (5).

In concluding I wish only to call attention to the need to examine the modeling problems of design and management of pavement systems in the light or, perhaps better, the darkness of uncertainty. The root of the problem is the notion of determinism-cause and effect, combined with the perversity of nature and man. For example, the input variables, traffic and environment, are clearly nondeterministic (stochastic) in the sense that one must attach a probability distribution to these inputs. Similarly, the pavement system itself, by virtue of its constituent materials and methods of construction, possesses a stochastic character; even a deterministic input to such a system will produce a stochastic output. Furthermore, application of a control likewise leads to an uncertainty in outcome.

How does all this uncertainty affect our efforts to develop a rational basis for design and management of pavement systems? Briefly, we can consider the previous apparatus used in this section only with reinterpretation of the primitive elements. We can define a discrete stochastic multistage decision process by asserting that a decision Y_n determines a set of possible outcomes (states) instead of a unique outcome. The state vector X_n is now a stochastic vector in the sense that its components are probability distributions. Furthermore, the transformation leading to state changes, i.e., G( ) in Eq. 10, is a stochastic transformation. In addition the criterion function, depending now on stochastic variables, is itself a stochastic quantity, as are the decision vectors, which depend on the system states. The condition of "minimum of the criterion function" can be replaced by minimum of the expected value of the stochastic criterion function, which leads to the notion of an optimal policy for a discrete stochastic process: Select a sequence of decision vectors $[Y_n(X_n)]$ such that the expected value of the criterion function is minimized. For the special case of a Markov process this problem has been studied in some detail (7). Whether a Markov model is adequate for the pavement system problem is another matter. This is an area needing considerable exploration.

SUMMARY

In this brief review of the status of research primarily associated with development of a more rational basis for design and management of flexible pavements, I have tried to emphasize two basic ideas: (a) the need to develop an overall structure for the entire pavement system, an assembly of many black boxes; and (b) the need to explore in some detail the contents of the various boxes to develop mathematical models of each subsystem, leading eventually to a model of the system in its entirety. A number of suggested directions in each of these categories have been discussed, and a more detailed treatment is given in other papers. I have separated the problem of design from the problem of management only for purposes of clarifying their treatment. One can and must eventually regard the two as one problem when more reliable models of system behavior become available, recognizing that this activity is a pattern recognition problem of special complexity.

What seems to me to be incumbent on those attending this workshop, as well as managers of research and development funds in general, is the development of a systems model for management of state and federal programs directed toward the problem under consideration. One needs to examine carefully the matter of "sensitivity" of various black boxes with regard to system performance. Although it may be commendable to study one box alone in the name of science, it is surely poor engineering practice to channel a lot of support to a subsystem with a low sensitivity factor vis-à-vis the total system performance. Such decisions regarding support should be made in the light of information and reason: They are difficult and agonizing, but that is what managers are expected to live with.

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DISTRESS MECHANISMS—GENERAL

B. F. McCullough

At the start, it is desirable to discuss the fundamentals of pavement behavior and performance. The conceptual pavement system illustrates the complex interrelationship that exists among material properties and the geometry (i.e., thickness) of the pavement layers, manifestations of pavement behavior, and pavement performance and failure. Thus, it is necessary to understand the interrelationship of these factors in order to establish concepts and procedures for improving components of the pavement system. The first item, material properties, is not included in this report, but the other two items, pavement behavior and pavement performance, are briefly discussed in the following sections.

PAVEMENT BEHAVIOR

The factors affecting pavement structural behavior have been defined and characterized in different ways by various individuals and groups (2 through 10). Although reasons for these characterizations may vary, it appears that the basic purpose in most cases has been to provide guidelines for design or evaluation. Such descriptions of pavement structural behavior have usually been formulated by defining factors that affect either pavement performance or pavement structure failure. A survey of the literature, however, indicates that there are no clear-cut and generally accepted failure definitions relating to some level of serviceability or performance and that there is no complete set of well-defined and generally accepted failure mechanisms for the pavement components.

In this study, an attempt has been made to associate material properties with modes of failure or distress through considerations of the various mechanisms and manifestations of distress. Limiting response (i.e., distress) modes have been divided into three categories: fracture, distortion, and disintegration. These are given in Table 1. With the exception of pavement slipperiness associated with the surface coefficient of friction, all forms of pavement distress can be related individually or collectively to these modes.

Also given in Table 1 are the manifestations of each mode of distress, together with a listing of the causes associated with each type of failure. Although the next logical step would be to list the pertinent material properties for each of the failure mechanisms noted, this has not been done because of the lack of suitable constitutive equations for materials and the lack of adequate failure theories. The first may be termed the primary manifestation, and those that occur progressively after it the secondary, tertiary, and so forth. The sequential order of these manifestations would vary depending on load, environmental conditions, and the like. In most cases, a number of these may occur simultaneously.

COMPARISON WITH AASHO MODEL

Technically, if the AASHO equation were all-encompassing, a mathematical model would be present for each of the distress mechanisms given in Table 1. Thus, a model would predict each of the distress modes of fracture, distortion and disintegration by inputing load, environment, construction, maintenance, and structural variables considering space and time. The AASHO model does not have this finesse in that it uses a gross transformation from the input components of a pavement structure, i.e., thickness and strength coefficients, to a present serviceability index (PSI). From

TABLE 1

MODES, MANIFESTATIONS, AND MECHANISMS OF TYPES OF DISTRESS

Mode	Manifestation	Mechanism
Fracture	Cracking	Excessive loading Repeated loading (i. e., fatigue) Thermal changes Moisture changes Slippage (horizontal forces) Shrinkage
	Spalling	Excessive loading Repeated loading (i. e., fatigue) Thermal changes Moisture changes
Distortion	Permanent deformation	Excessive loading Time-dependent deformation (e. g., creep) Densification (i. e., compaction) Consolidation Swelling
	Faulting	Excessive loading Densification (i. e., compaction) Consolidation Swelling
Disintegration	Stripping	Adhesion (i. e., loss of bond) Chemical reactivity Abrasion by traffic
	Raveling and scaling	Adhesion (i. e., loss of bond) Chemical reactivity Abrasion by traffic Degradation of aggregate Durability of binder

prediction of PSI, a performance history can be obtained and failure of the system can be evaluated in terms of a minimum serviceability level and total dollar cost to the system. The performance of the pavement is a measure of the accumulated serviceability provided by the facility and may be expressed as a direct function of the present serviceability history for the pavement (2). A second model is a structural number model that was also developed at the Road Test (11) and subsequently used by the AASHO Design Committee to formulate the Interim Guides (12, 13, 14). These models are expressed as follows:

$$p = 5.03 - 1.91 \log(1 + \overline{SV}) - 0.01\sqrt{C + P} - 1.38\overline{RD}^2 \tag{1}$$

where

p = present serviceability index,
$\overline{SV}$ = mean slope variance, a summary statistic of wheelpath roughness,
C = area of detrimental cracking per 1,000 sq ft,
P = area of patching per 1,000 sq ft, and
$\overline{RD}$ = average rut depth in the wheelpath.

$$P(\underline{x}, t) = \underset{s=0}{\overset{s=t}{F}} \left[p(\underline{x}, s)\right] \tag{2}$$

where

$P(\underline{x}, t)$ = performance as a function of space and time,
t = time, and
$\underline{x}$ = position vector of a point referred to a coordinate system.

$$SN = A_1D_1 + A_2D_2 + A_3D_3 \tag{3}$$

where

SN = structural number of system,
A_i = structural coefficient of the ith layer, and
D_i = thickness of the ith layer.

$$p = P_1 - \left(\frac{W}{\rho}\right)^{\beta} \tag{4}$$

where

P_1 = initial PSI,
W = number of equivalent wheel loads, and
β, ρ = parameters depending on layer thickness and strength coefficient and wheel load magnitude and configuration.

The AASHO equation uses a structural number value (Eq. 3) to compute the present serviceability value (Eq. 4) at the end of a stated time or traffic period. The computed performance at the end of the traffic period does not indicate the relative magnitude of cracking, patching, slope variance, and rut depth (Eq. 1). Rather, some function of their combined values will be equal to the computed PSI at time t. Because these mathematical models are equations statistically derived from AASHO Road Test data, they may be applied successfully within the limits of material types and thicknesses and experiments at the AASHO Road Test. The use of any new materials may be an extrapolation of the equation beyond its boundary conditions; hence, the applicability is questionable and remains to be verified. Thus, one immediate improvement of the AASHO model would be to quantify Eqs. 2, 3, and 4 for fracture distortion and disintegration and their combined value of distress index (Eq. 1) on the basis of theory. With such a model, PSI could be predicted on the basis of the actual output information, i.e., fracture, disintegration, and distortion, rather than through a gross transformation between input variables and performance based on field observations. Such models would make it possible to design for any material and any conditions.

FUNCTIONAL MODELS

The complete quantification of all of the distress manifestations and mechanisms is an extensive undertaking. Therefore, the approach used here is to show a logical method for quantifying several of the distress mechanisms and to demonstrate their applicability in the systems model.

Fatigue or repeated loading has been the subject of considerable research (15 through 23) with the result that there is a great deal of information available for use. Therefore, this distress mechanism has been selected for quantification. The complex interaction of the various distress mechanisms and manifestations was discussed previously. Figure 1 shows the interrelation of several distress manifestations that may be related with load repetitions. In this example, the distress mechanism of repeated loading leads to a primary manifestation of cracking, and the continued repeated load application leads to a secondary manifestation of spalling (fracture) and permanent deformation (distortion). The cracking of the pavement structure may also reduce the load-carrying capacity of the pavement structure, and for the same loads a secondary mechanism of excessive loading results and leads to a secondary distress manifestation of faulting (distortion) and spalling (fracture). In this manner, the initial distress mechanism of repeated loading has caused the distress modes of fracture and distortion to occur in the pavement structure. It is easy to envision the complex interractions that would develop when several distress mechanisms are involved.

The possible progressive development of the distress index due to an initial effect of the distress mechanism of repeated loading and the resulting secondary distress mechanisms is shown in Figures 2 through 5. In Figure 2, the development of the cracking index is conceptually shown in terms of traffic application. For the first period of traffic applications, little or no cracking occurs. When traffic reaches a value of n_c, pro-

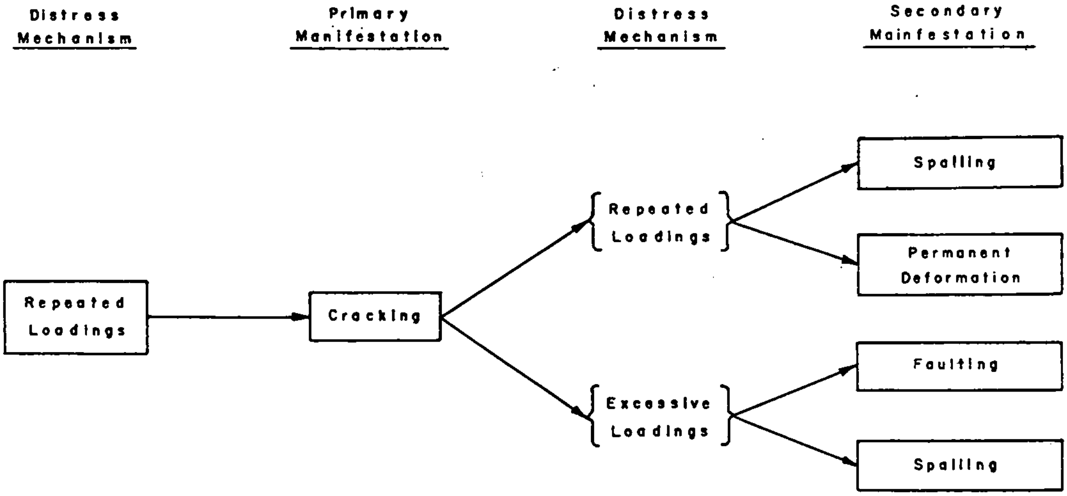

Figure 1. Interrelationship of distress mechanisms and manifestations.

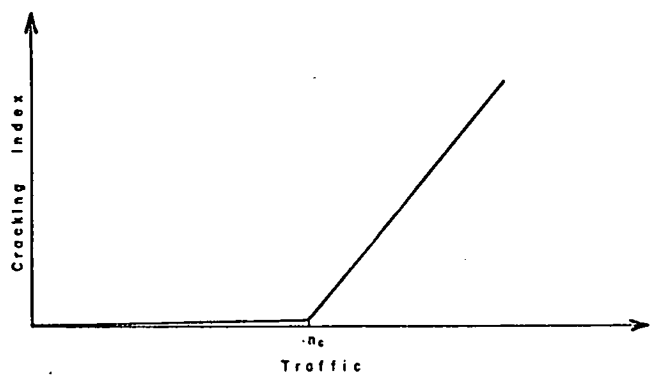

Figure 2. Progressive development of cracking index with traffic.

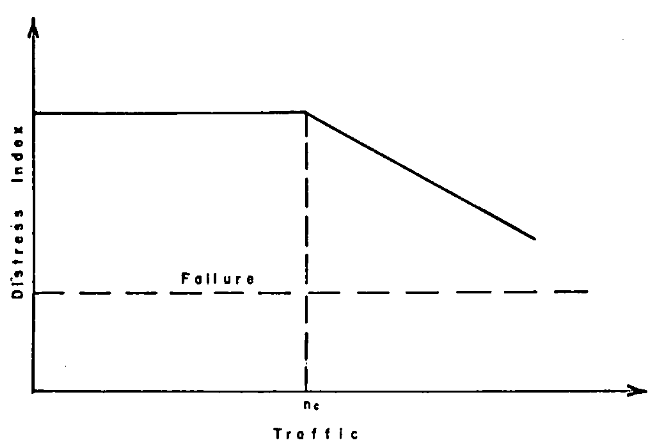

Figure 3. Progressive development of distress index considering cracking.

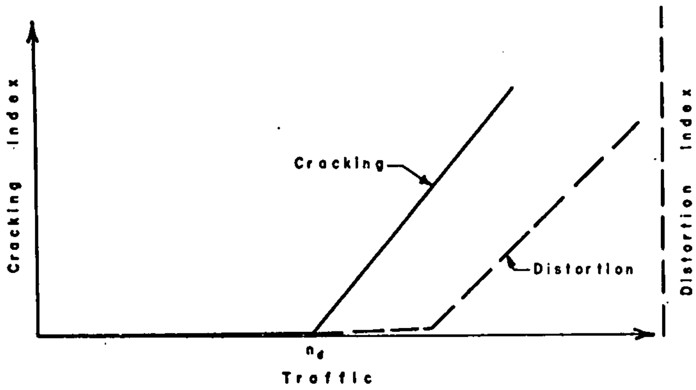

Figure 4. Progressive development of primary cracking and secondary distortion.

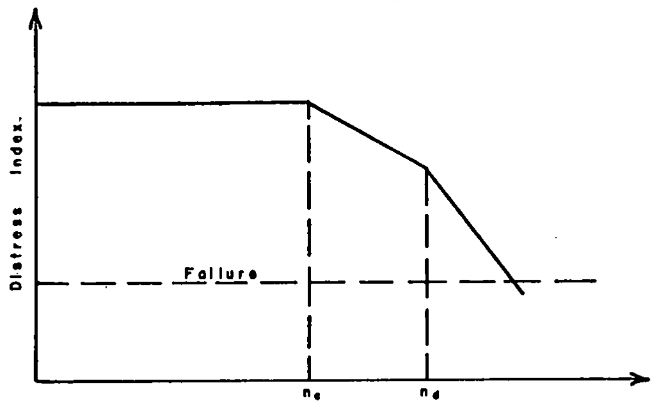

Figure 5. Progressive development of distress index considering both cracking and distortion.

gressive cracking begins, and the cracking index increases rapidly. If it is assumed that only the fracture mode occurs, the cracking index is used with Eq. 6 (Appendix) to compute the distress index. Its history is shown in Figure 3. There is no change in the index until cracking occurs at n_c traffic applications, at which time a progressive decay commences.

The development of the secondary manifestations of faulting and permanent deformation is shown in Figure 4 in terms of the distortion mode of distress. The distortion index might begin at a traffic value n_d, which is greater than n_c inasmuch as distortion is a secondary manifestation in this case. The shape of the distress function changes with the addition of distortion (Fig. 5), and the decay or slope of the distress index will be greater when the secondary manifestations of distortion and fracture occur, because of their compounding effect.

Failure of the system occurs when the distress index decreases below a minimum acceptable value. The preceding discussion illustrates the functional concepts involved in quantifying the distress index. The next step is to utilize the necessary constitutive equations to solve the functional equations.

SELECTION OF BOUNDARY VALUE PROBLEMS AND CONSTITUTIVE EQUATIONS

Detailed steps for characterizing materials and using the results in boundary value problems are discussed in other papers in this report. As a precursor to such complex improvement, this example illustrates the application of the best developed constitutive equation and boundary value problems in the present state of the art. The constitutive equation for linear elastic theory (24) and layered theory (25, 26) probably represent the most advanced state of the art available for use at the present time.

Figure 6 shows a typical pavement structure cross section and the elastic parameters, i. e., modulus of elasticity and Poisson's ratio, and the pavement geometry value, i. e., thickness, required in the layered system program. These values are used with the layered program to compute the mechanical state of stress and strain in the pavement structure. These computed values may then be compared with the corresponding limiting values to predict cracking. If the computed stress is greater than the strength, then cracking is assumed to occur.

The computed values of stress and strain are deterministic in nature inasmuch as the input values are considered to be exact quantities. Thus, a deterministic solution does not consider the possibility of variations in properties, as required by conceptual Eq. 6. With the absence of stochastic concepts, a deterministic approach implies that, when the stress is greater than the strength, failure will occur at every point in the pavement where a wheel load causing the limiting stress passes over. Of course, experience and studies show that cracking does not occur in this manner but, rather, on a progressive basis (11). Thus, it is necessary that the stochastic concepts be injected into the approach in order to predict progressive cracking more accurately. One method previously developed assumes that, if the stress is independent of strength, the probability of distress may be stated as

$$P\{C\} = P\{\text{stress} > b\} + P\{\text{strength} < b\} \tag{5}$$

where

$P\{\}$ = probability of an event occurring, and
b = a value defining the point where stress and strain values overlap.

A stochastic equation permits one to quantify this functional notation. The use of Eq. 5 allows the percentage of surface area of a roadway experiencing cracking to be predicted for certain stress and strength variations around the mean value shown.

In addition to these properties, the fatigue characteristics of the materials are an input property required in predicting cracking due to the repeated load distress mechanism. A typical fatigue curve for portland cement concrete and asphalt concrete (Fig. 7) indicates that, the greater the stress level is, the fewer will be the number of load repetitions required to failure. The solid line in Figure 8 is an average fatigue line for the data. Monismith, Kasianchuk (27), and others have shown that the stochastic variation in

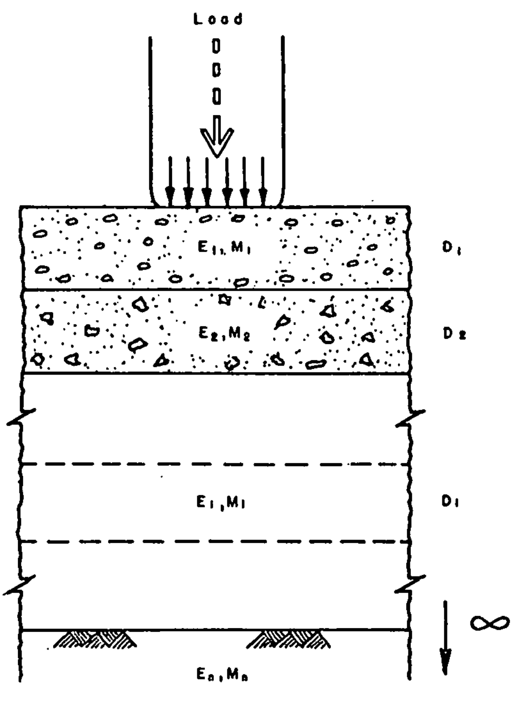

Figure 6. A typical pavement structure cross section showing the elastic parameters.

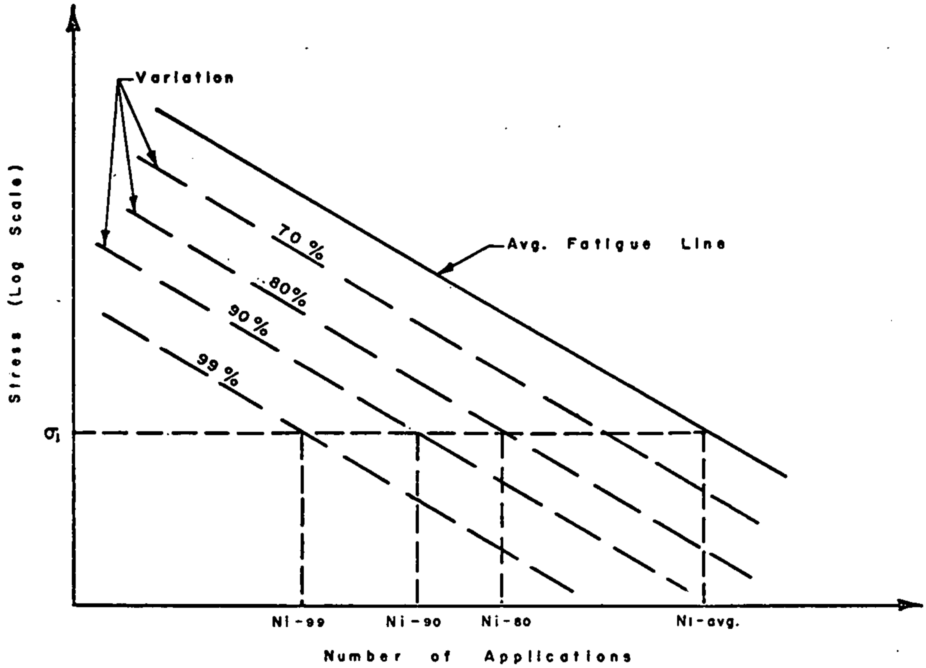

Figure 7. Typical fatigue for a pavement material.

asphalt concrete may be described by lines parallel to the average fatigue line as shown in Figure 7. Each line indicates the probability that a pavement subjected to a given stress level will last a given number of applications. In essence, this principle implies that, for a given stress level, the less risk of cracking one is willing to take, the smaller will be the allowable number of load repetitions. For example, the ith stress level will go N_i - 99 applications with a probability of 99 percent, i. e., only 1 percent chance of failure. However, if the user is willing to accept the probability of 20 percent failure, then the material will last N_i - 80 load repetitions, which is greater than N_i - 99 (28).

SUMMARY

In this report, the feasibility of using research findings and results to improve a systematic pavement design procedure is demonstrated.

The conceptual sequence for modifying the gross transformation between input variables and performance may be developed as follows:

1. Predict a distress manifestation based on a primary distress mechanism (Table 1).
2. Note that the occurrence of a primary distress manifestation leads to the initiation of a secondary distress mechanism which in turn leads to secondary distress manifestations. This process may occur for several additional levels, i. e., secondary, tertiary, and so on (Fig. 1).
3. Define the effects of the primary, secondary, and higher order distress mechanisms, and combine them to predict a distress index history, i. e., performance, for the pavement (Fig. 5).

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APPENDIX

DISTRESS INDEX

$$\underline{DI}(\underline{x}, t) = \underset{s=o}{\overset{s=t}{\underline{F}}} \left[\underline{C}(\underline{x}, t),\ \underline{S}(\underline{x}, t),\ \underline{D}(\underline{x}, t)\ \underline{x}, t\right] \qquad (6)$$

where

t = time;
$\underline{x}$ = position vector of a point referred to a coordinate system (space);
$\underline{DI}(\underline{x}, t)$ = distress index, a matrix function of space and time;
$\underline{C}(\underline{x}, t)$ = measure of fracture, a matrix function of space and time;
$\underline{S}(\underline{x}, t)$ = measure of distortion, a matrix function of space and time; and
$\underline{D}(\underline{x}, t)$ = measure of disintegration, a matrix function of space and time.

$\underline{C}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural $\overline{\text{variables}}$ and of space and time.

$\underline{S}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural $\overline{\text{variables}}$ and of space and time.

$\underline{D}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural $\overline{\text{variables}}$ and of space and time.

$\underline{DCI}(\underline{x}, t)$, or Decision Criteria Index, is a function of riding quality, economics, $\overline{\text{safety}}$, maintainability, and other factors and of space and time.

OBSERVATION OF DISTRESS IN FULL-SCALE PAVEMENTS

Fred N. Finn

Pavement design principles are commonly indicated to be based on empirical procedures or on empiricism. The dictionary defines the word "empirical" as (a) relying on experience or observation alone, often without due regard for system and theory, (b) originating in or based on observations or experience, (c) capable of being verified or disproved by observation or experience. The key words in these definitions are observation and experience. The implication of this emphasis on the use of empirical procedures is that agencies have been accumulating observations on which current material, construction, and thickness design concepts are based.

With no disrespect intended to those agencies that have been working in this area, it is doubtful if extensive systematic and continuous observations of pavement performance have been accumulated in this country. The literature contains reports of extensive investigations that attempt to define and describe pavement performance and to describe types and causes of distress. Remarkably, very little of this kind of information has found its way, except possibly by inference, into research reports dealing with developments that depend on performance as the dependent variable.

Specific road tests are probably prime examples of the use of systematic and continuing observations to develop a dependent, numerical value from which conclusions are drawn and documented. The WASHO and AASHO projects would be the most widely known examples of the use of empiricism in developing our technology. Investigators in Canada have used field observations of distress (frequency of transverse cracks) as a dependent variable in trying to develop empirical and theoretical correlations to thermal cracking. Most researchers would like to believe that observations of performance can be related to theory, and it is for this reason that a workshop was held.

As is implied by the third definition of empirical, observations and experience work two ways. In the first instance, it should be possible to determine what the priority needs are in terms of performance. For example, if observations indicate that pavements lose serviceability as a result of a specific type of mechanism, say plastic deformation or distortion, then it should be clear that research should be directed toward solving this problem. When the research is completed and the results are implemented, it will be possible to determine whether a satisfactory solution was obtained, more work is required, or a satisfactory solution has been achieved with a new problem being induced by the solution. Researchers need to know that the problems they are trying to resolve are real and significant. This can only be determined by systematic and continuous observations of performance of full-scale pavements.

The pavement is an excellent example of the "black box" description of systems engineering. Observations of distress represent a summation of everything, including the variables of materials and construction and the influence of environment, age, and traffic. It is not an easy matter to look at a pavement and conclude precisely what mechanisms are involved or are in control. It is not easy, but it is a beginning. Given enough observations over wide enough areas and long enough times, some patterns can surely evolve and have evolved.

PROCEDURE

It is important to mention briefly something about present techniques for evaluating the condition of a pavement. Observations or condition surveys are made on the basis of subjective evaluations that are primarily aimed at identifying the type of distress and the level of severity. To evaluate the cause of distress by such techniques has proved somewhat elusive and unreliable except when accomplished by experienced and highly knowledgeable observers. Even then, serious differences of opinion can occur. Probably if more effort had been expended in this area of study, fewer differences would exist and better associations would be possible. For the moment, it is not unlike the doctor who is allowed to look at the patient and possibly feel the patient but is not allowed to perform any tests before making his diagnosis. Thus, present subjective techniques allow the observer to make certain statements about the types of distress (cracking, distortion, and disintegration) and considerably fewer about the causes of distress.

Efforts by a number of agencies are being made to introduce objective measurements into observations of performance. Factors such as riding quality, deflection, surface texture, and coefficients of friction have been or are being considered as supplemental information to assist in defining the causative factors related to loss in serviceability or to occurrence of observable distress.

Lack of uniform terminology to describe pavement performance has not helped in trying to define or to communicate the condition of a pavement or the type of distress being observed. The same type of distress may be described differently or with less emphasis by engineers in various parts of the country or by different agencies. Highway Research Board Special Report 113 attempts to standardize nomenclature for pavement components and deficiencies. It is to be hoped that terminology suggested in that publication can serve as a useful beginning in minimizing some of our communications problems.

TYPES OF DISTRESS

Although complete unanimity does not exist in the literature as to the types of distress, there is some consensus that distress can be grouped into three classes: (a) fracture, (b) distortion, and (c) disintegration. The input factors required to bring about these types of distress become very complicated and involve traffic loading, environment, materials properties, construction, and geometrics (layer thickness). Table 1 in McCullough's paper gives a further breakdown of the possible distress mechanisms. Unfortunately, there is probably a high level of interaction between each observable distress and the mechanisms involved.

Identifying the three types of distress is not difficult, and even this amount of information would be helpful in establishing research priorities. The next major definition or differentiation would be between traffic-associated and non-traffic-associated distress. Traffic-related distress can usually be related to a location within the paved area. If distress occurs or is confined to the areas subjected directly to vehicular loading, it is probably traffic-associated. If it does not appear to be associated with areas in direct contact with loads, it is probably an environment, construction, or materials problem. Obviously, this areal association is an oversimplification, but it should help in estimating the mechanism of distress, which, in turn, should help in defining the type of research needed.

OBSERVATIONS OF DISTRESS

A review of the literature will give the impression that an abundance of distress exists on the highway system. This is to be expected inasmuch as the serviceability of pavements is known to decay with time and traffic. Many pavements observed to be exhibiting distress have simply outlived their design lives and have not received the anticipated amount of rehabilitation. Some pavements exhibit distress somewhat prematurely because the reliability factor of present design methods is, and should be, something less than 1.0. It is the role of research to improve, quantify, and control the reliability factor in order to provide the most economic balance between performance requirements and costs.

There is no single source of information that permits highway technologists to summarize how full-scale pavements perform. The author has been involved in several projects concerned with pavement performance and observations of distress. It is not suggested that these observations are a rigorous evaluation of how pavements perform; however, the author has made systematic evaluations in 22 states and the District of Columbia that involved more than 150 projects of widely varying designs, uses, materials, construction procedures, and environments. Also, the author has had the opportunity to observe the biweekly performance of the asphalt test sections on the AASHO Road Test during the 2-year traffic period. On the basis of these observations, the author has permitted himself to make some statements, however unreliable, about the occurrence of pavement distress on full-scale pavements.

The questions are asked: How do asphalt pavements perform? What kind of distress is most commonly observed? What might be the mechanism involved? These questions will be partially answered by a summary of the results of three rather systematic pavement evaluation studies. Following this, results reported by other agencies will be used in an effort to show the present performance situation.

A statewide pavement condition survey, involving pavements ranging from 3 to 8 years in service life, in one of the northwestern states in 1966 indicated the following:

1. An observable amount of distortion was occurring on 23 percent of the pavements with 3 percent having progressed to the stage where some maintenance would be indicated;
2. Minor amounts of disintegration had occurred on 25 percent of the projects, all of which could be adequately maintained by the application of surface seals;
3. Fracture was observed on 63 percent of the projects, 31 percent of which would be scheduled for programmed maintenance; and
4. Transverse cracks were noted on 60 percent of the projects, 13 percent of which would be eligible for programmed maintenance.

The riding quality of pavements with transverse cracks was only 0.4 PSR less than pavements without transverse cracks. On the basis of this survey it would be concluded that traffic-associated cracking is of major concern not only in terms of the amount but also in terms of the cost to maintain and rehabilitate.

The second pavement condition survey was part of a Bureau of Public Roads investigation to determine the changes in physical and chemical properties of asphalt and to attempt to evaluate the role of asphalt in the performance of pavements. The investigation included limited but systematic observations of performance in 18 states and the District of Columbia. All of the projects were 11 to 12 years old at the time of the survey and included Interstate highways and local roads and streets. Of the projects observed, 20 percent exhibited traffic-associated fracture at a level suggesting immediate or programmed maintenance. Only 4 percent of the pavements exhibited rutting at a level that could require maintenance, although most of the pavements exhibited some longitudinal distortion (not exceeding $\frac{1}{4}$ in. average). About 20 percent exhibited surface disintegration indicating the need for immediate or programmed maintenance.

The Sweetwater Test Road in San Diego County is a cooperative project to evaluate the structural capability of eight base types constructed at four levels of thickness. It is significant to discuss the performance of these sections after 4 years of service. None of the sections has exhibited distortion in any significant amount. All of the sections exhibited disintegration, which has been corrected by a slurry seal. Based on observations by the author, 27 percent of the sections exhibit traffic-associated cracking. According to the experiment design, it would be expected that from 25 to 50 percent of the sections would exhibit distress during the 4-year time period. It is significant that much of the distress that has occurred is in the form of traffic-related fracture. It is pertinent to comment that on the Sweetwater Test Road, longitudinal non-traffic-associated cracking is occurring along the outer edge of the paved area. This cracking has made the identification of mechanisms very difficult; nevertheless, some associations are believed to be justified. On the basis of these observations, the following conclusions are suggested:

1. Traffic-associated cracking is one of the first indications of distress observable on asphalt pavements;

2. Traffic-associated cracking can occur with little (less than ¼ in.) surface distortion; and
3. Traffic-associated cracking does not appear to be highly correlated with surface disintegration.

It should be mentioned that one class of distress considered by many, especially in certain northern tier states and in Canada, to be important has not been emphasized by the author—specifically, transverse cracking, believed to be caused by thermal stresses. Although this type of distress has been observed by the author, it has not appeared to be highly significant to the overall performance. Canadian investigators (McLeod, Haas, Anderson, and others) have conducted extensive observations, both systematic and continuous, that strongly indicate that thermal cracking can be a very important design factor for asphalt pavements in particular environments. Several test projects are under way in Canada to determine methods to control this particular type of distress. It must be conceded, therefore, that thermal cracking can be sufficiently important to be assigned a high priority for research needs in certain environments.

One of the most extensive investigations of pavement performance was made in connection with the AASHO Road Test. Possibly the single most significant contribution of this project was the development of a numerical index of serviceability in terms of riding quality. Riding quality, per se, is a summation of everything that can happen to a pavement by all of the mechanisms that will cause a reduction in traveling comfort. In addition to riding quality determinations, considerable effort was made to study the occurrence of distortion in the transverse profile and the occurrence of load-associated cracks. No significant amount of non-load-associated cracking was recorded in the 2-year traffic period. The results of these studies are particularly pertinent to this workshop.

Highway Research Board Special Report 61E summarizes the performance measurements from the AASHO Road Test. The following conclusions about changes in transverse profile are suggested by that report:

1. Nearly all of the sections exhibited some changes in transverse profile at the completion of traffic testing.
2. Profile changes were due principally to decreases in thicknesses of component layers. Although this conclusion may be tenuous for the thinner sections, the evidence was reasonably conclusive for the sections of substantial thickness. Some evidence indicated that sections exceeding 11 in. in thickness exhibited only minor distortion in the underlying layers.
3. Lateral movement of materials was the primary cause of change in thickness. (The author wants to point out that the rate of change in the transverse profile decreased markedly after the first 300,000-lb axle load applications. This would suggest some stabilizing mechanism with time or traffic. Possibly the increase in density occurring in the structural components was causing significant increases in strength that, in turn, reduced the tendency for lateral movements.)
4. A reduction in the amount of change in transverse profile was observed when a stabilized base, either asphalt or cement, was incorporated into the section.

The following conclusions are given for cracking:

1. More cracking occurred during periods when the pavement structure was in a relatively cold condition than when the pavement was warm. (This conclusion becomes rather crucial to the development of damage criteria and is complicated by a lack of information relative to when and where cracks first occur and how they propagate.)
2. Cracking and the extent of change in transverse profile were related.
3. The occurrence of cracking and deflection were related.
4. Cracking and pavement thickness were related.

These suggested conclusions may or may not be truly representative of how pavements will perform if different materials had been used or even if different construction requirements had been specified or if the project had been located in a different environment. They were, nevertheless, indicative of what happened at the AASHO

Road Test and, as such, must be considered to be representative of how some pavements perform. The results from this project would indicate that technology that would reduce, control, or reasonably predict the potential for changes in transverse profile, or for cracking, would be a substantial contribution to the present state of knowledge.

SUMMARY

Based on the observations of the author and observations that have been reported by others, it would appear that traffic-associated cracking would be the number one priority item for improving and extending the performance of asphalt pavement. Also, the systems approach to solving the problem, which implies taking into consideration all of the interactions of materials, construction, maintenance, and environment, should be basic to the solution of such problems. Finally, the solution to traffic-associated cracking problems should not create problems in the distortion or disintegration categories.

Investigators are urged to consider implementing systematic and continuous observation of pavement performance as a prime determinant in establishing research priorities and to confirm the effectiveness of having implemented research findings. Research should have an assignable payoff, and only by observing performance can such a payoff be quantified.

FUNDAMENTALS OF MATERIAL CHARACTERIZATION

Russell A. Westmann

In predictions of the structural performance of an existing or proposed pavement, it is common to consider the system to be constructed of several different homogeneous materials. Each material is modeled as a continuum, and certain mechanical properties are ascribed to the material components of the pavement. The stress and displacement fields in the model are then calculated by using the principles of solid mechanics. To obtain the desired estimates of the structural behavior of the pavement system, we examine the stress and displacement fields in light of possible failure laws of the material and failure mechanisms of the system. This last step involves the failure analysis of the pavement.

Considerable advances have been made in numerical solution techniques for boundary value problems in solid mechanics. Using modern methods such as finite elements makes it possible to solve problems for a wide class of geometries and material properties. The question now is not how to solve the boundary value problems but how to state accurately the material properties so that the boundary value problems can be formulated.

After this is done the stress and displacement fields can be accurately predicted, and attention can be directed to the failure analysis. Unfortunately, experience in this area is limited. At this time, relatively little is known about the circumstances that lead to material failure. In addition there have been few attempts to relate the stress and displacement fields to the pavement performance through an appropriate failure analysis.

It is the purpose of this paper to outline some of the fundamentals of material characterization. For the purposes of presentation, this outline is divided into two parts—constitutive laws and material failure or fracture. It is hoped that with the aid of this paper the reader can assess the state of the art and see what research must be performed to improve the mechanical characterization of highway pavement materials.

BASIC DEFINITIONS

The stress and deformation analysis of pavement systems is based on a modeling of the actual structure. To interpret the results of such an analysis requires an understanding of the model and its relation to the real problem. The usual approach is to model the real structure by a continuum and then focus attention only on the statistical stresses and deformations. In this way the effects of heterogeneity are ignored except at a global level.

The "measure of deformation" of the continuum is the strain tensor, ϵ_{ij}. [For purposes of brevity, index and summation notation are used (11). Where feasible, the equations are partially expanded in conventional engineering notation to improve clarity.] In terms of the displacements, μ_i, of the continuum, this is given by

$$\epsilon_{ij} = \frac{1}{2}\left(\frac{\partial \mu_i}{\partial x_j} + \frac{\partial \mu_j}{\partial x_i}\right) \quad (i, j = 1, 2, 3)$$

or

$$\epsilon_{xx} = \frac{\partial \mu_x}{\partial x}$$

$$\epsilon_{xy} = \tfrac{1}{2}\left(\frac{\partial \mu_x}{\partial y} + \frac{\partial \mu_y}{\partial x}\right) \tag{1}$$

These are valid provided the displacement gradients may be neglected compared to unity, i.e., $|\frac{\partial \mu_i}{\partial x_j}| << 1$. This latter restriction is usually fulfilled in pavement problems.

The "transfer of force" in the continuum is measured in terms of the stress tensor, τ_{ij}. The stress field must satisfy equilibrium equations that, in the absence of inertia effects, are

$$\frac{\partial \tau_{ij}}{\partial x_i} + F_i = 0$$

$$\frac{\partial \tau_{xx}}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} + F_x = 0 \tag{2}$$

where $F_i(F_x)$ is the distributed body force acting on an element of the body. This classical form of the equilibrium equations is valid provided certain restrictions on the displacement and stress fields are satisfied.

The complete statement of the continuum problem is then made upon giving the relationship between stress and strain in the material, i.e., the "constitutive law." In general this might be stated as

$$\epsilon_{ij} = F_{ij}(\tau_{lm}, x_l, t, T)$$

or

$$\epsilon_{xx} = F_{xx}(\tau_{xx}, \tau_{yy}, \ldots; x, y, z; t, T) \tag{3a}$$

and

$$\tau_{ij} = G_{ij}(\epsilon_{lm}, x_l, t, T) \tag{3a}$$

$$\tau_{xx} = G_{xx}(\epsilon_{xx}, \epsilon_{yy}, \ldots; x, y, z; t, T) \tag{3b}$$

where t and T denote the time and temperature variables respectively. Equation 3a (3b) simply states that the strain (stress) at a point is a function of stress (strain), time, temperature, and position of the point.

Prescription of the problem geometry and suitable boundary conditions then completes the statement of the boundary value problem. Analytical or numerical solution of this mathematical problem yields the stress and deformation fields and the desired engineering predictions. The accuracy of these predictions depends on the accuracy of the model. The most serious approximation made in the modeling of the real pavement occurs in the selection of the stress-strain relation.

The accuracy attainable in modeling the material properties depends on the complexity of the material response. Accordingly, materials are categorized by their primary response characteristics. The classification used in the following depends on whether the material response is linear or nonlinear and rate-dependent or rate-independent.

It is first necessary to define what is meant by linear and rate-independent. These are best interpreted in terms of the time-dependent input, P(t) (load) and measured time-dependent response, $\Delta(t)$ (strains and deflection), of the structure or material specimen.

Assume that for given inputs $P_1(t)$ and $P_2(t)$ the measured responses are $\Delta_1(t)$ and $\Delta_2(t)$ respectively. Symbolically this may be written

$$P_1(t) \longleftrightarrow \Delta_1(t)$$

$$P_2(t) \longleftrightarrow \Delta_2(t)$$

A material is said to be linear if for an input $c_1P_1(t) + c_2P_2(t)$ the measured response is $c_1\Delta_1(t) + c_2\Delta_2(t)$ for all real numbers c_1, c_2; i.e.,

$$C_1P_1(t) + C_2P_2(t) \longleftrightarrow C_1\Delta_1(t) + C_2\Delta_2(t)$$

Note that this implies that superposition holds; in particular, quantities can be superposed at different times. If the response corresponding to a particular input P(t) is known, then, because of the linearity of the material, the response for any input constructed from P(t) can be calculated by superposition.

A material is rate-independent if, for an input P(t) with measured response $\Delta(t)$, an input P(ct) has response $\Delta(ct)$ for all real numbers c.

No material exists that is linear or rate-independent for all magnitudes and frequencies of loads. In actuality, linearity and rate independence will only hold for restricted ranges of the loadings. It is always an experimental problem to determine the extent of these regions.

One additional term should be defined. Elastic means that all of the deformation is recoverable and, upon removal of the load, the material returns to its initial state retracing the loading path. It should be noted that elastic materials can still be nonlinear.

The next sections consider progressively more sophisticated stress-strain relationships. Starting with linear, rate-independent materials, various constitutive laws are presented including a model for nonlinear, rate-dependent behavior.

LINEAR, RATE-INDEPENDENT MATERIAL

In the following it is assumed that it has been experimentally verified that the material is linear and rate-independent for the stress levels of practical interest.

Linear, Elastic, and Isotropic

The best known constitutive equation is a generalized Hooke's law for isotropic materials,

$$\epsilon_{ij} = \frac{(1+\nu)}{E}\tau_{ij} - \frac{\nu}{E}\tau_{ii}S_{ij}$$

$$\epsilon_{xx} = \frac{\tau_{xx}}{E} - \frac{\nu}{E}(\tau_{yy} + \tau_{zz})$$

$$\epsilon_{xy} = \frac{(1+\nu)}{E}\tau_{xy} \tag{4}$$

where E and ν are Young's modulus and Poisson's ratio respectively. S_{ij} is Kronecker's delta, which equals one when i = j and zero when i $\neq$ j. For the material to be isotropic, it is necessary that the response of a material specimen be independent of its original orientation in the parent material. To verify isotropy, we should run a series of tests on specimens with initially different orientations. If the response is the same for all of these, then the material is isotropic.

Once isotropy is verified, the standard test usually performed is the uniaxial tension or compression test. Measurement of the axial strains and the lateral contractions or expansions for the linear part of the stress-strain curve then permits evaluation of the material constants E and ν.

This is not a complete characterization of the material. Although one-dimensional linearity has been tested and the material constants have been measured, it is still necessary to determine whether superposition holds for three-dimensional stress states. This necessarily requires three-dimensional tests. An example of a suitable experiment is the superposition of a shearing stress on an initial hydrostatic state. Tests of this type serve to verify (or disprove) the validity of Eq. 4 and indicate its range of applicability for three-dimensional states.

Aside from the obvious feature of linearity, it should be pointed out that Eq. 4 indicates that a pure shearing stress causes only a pure shearing strain. In addition, a hydrostatic stress, $\tau_{xx} = \tau_{yy} = \tau_{zz} = P$, produces only a volume change, $\Delta V = \epsilon_{xx} + \epsilon_{yy} + \epsilon_{zz} = [(1 - 2\nu)/E]3P$, without inducing any shearing strain. Only a linear, isotropic material exhibits both types of behavior.

Finally, material properties might be different for uniaxial tensile and compressive states. It is obvious what tests might be performed to detect this feature. Of course, if the properties are different in tension and compression, then the preceding constitutive law, Eq. 4, is not suitable because the material is not linear.

Linear, Elastic, and Anisotropic

In the case of a general linear, anisotropic material, each component of strain depends in a linear manner on all the components of stress (standard matrix notation is used here for convenience)

$$\begin{Bmatrix} \epsilon_{xx} \\ \epsilon_{yy} \\ \epsilon_{zz} \\ \epsilon_{yz} \\ \epsilon_{xz} \\ \epsilon_{xy} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & S_{14} & S_{15} & S_{16} \\ S_{21} & S_{22} & S_{23} & S_{24} & S_{25} & S_{26} \\ S_{31} & S_{32} & S_{33} & S_{34} & S_{35} & S_{36} \\ S_{41} & S_{42} & S_{43} & S_{44} & S_{45} & S_{46} \\ S_{51} & S_{52} & S_{53} & S_{54} & S_{55} & S_{56} \\ S_{61} & S_{62} & S_{63} & S_{64} & S_{65} & S_{66} \end{bmatrix} \begin{Bmatrix} \tau_{xx} \\ \tau_{yy} \\ \tau_{zz} \\ \tau_{yz} \\ \tau_{xz} \\ \tau_{xy} \end{Bmatrix} \tag{5}$$

where the material matrix $[S_{ij}]$ is symmetric. As a result, there are 21 material constants that must be determined if a material exhibits no symmetry. No effort is made here to describe what these tests might be, but reference may be made to Hearmon (1).

From Eq. 5 it is apparent that there is coupling between the voluminal behavior and shearing behavior of the material. For example, application of a pure tensile stress, $\tau_{xx} = T$, $\tau_{yy} = \tau_{zz} \ldots = 0$, results in all components of the strain having nonzero values:

$$\epsilon_{xx} = S_{11}T, \ \epsilon_{yy} = S_{12}T, \ \ldots, \ \epsilon_{xy} = S_{16}T$$

Some materials exhibit symmetry with respect to planes in the body. In these cases, the number of material constants reduces somewhat. For example, an orthotropic material has three (perpendicular) planes of symmetry. If the coordinate system coincides with these planes, Eq. 5 may be written as

$$\begin{Bmatrix} \epsilon_{xx} \\ \epsilon_{yy} \\ \epsilon_{zz} \\ \epsilon_{yz} \\ \epsilon_{xz} \\ \epsilon_{xy} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & 0 & 0 & 0 \\ S_{21} & S_{22} & S_{23} & 0 & 0 & 0 \\ S_{31} & S_{32} & S_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & S_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & S_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & S_{66} \end{bmatrix} \begin{Bmatrix} \tau_{xx} \\ \tau_{yy} \\ \tau_{zz} \\ \tau_{yz} \\ \tau_{xz} \\ \tau_{xy} \end{Bmatrix} \tag{6}$$

Taking into account the symmetry of the material matrix, we see that there are only nine material constants.

For the special case where every plane is a plane of symmetry, i.e., isotropic material, $S_{11} = S_{22} = S_{33} = 1/E$, $S_{12} = S_{13} = S_{23} = -\nu/E$, and $S_{44} = S_{55} = S_{66} = (1 + \nu)/E$, which, as in Eq. 4, depends on only two constants.

In the remainder of the paper attention will be focused entirely on isotropic materials. In many cases the results can be extended to include anisotropic effects.

LINEAR, RATE-DEPENDENT MATERIAL

In the following it is assumed that suitable tests have been performed to ascertain that the material is linear, isotropic, and rate-dependent.

Uniaxial Characterization

The common test is the uniaxial tension or compression creep test. In this test a load of magnitude σ_0 is instantaneously applied to the at-rest specimen and held constant with respect to time. The resulting time-dependent strain, $\epsilon(t)$, is measured, and the normalized result, $J(t) = \epsilon(t)/\sigma_0$, forms the creep curve or creep function.

In the case of the relaxation test, an instantaneous strain, ϵ_0, is enforced and maintained constant with respect to time. The resulting uniaxial stress in the specimen then relaxes with respect to time. The normalized curve, $G(t) = \sigma(t)/\epsilon_0$, is termed the relaxation function.

Other tests used are the constant strain rate and steady-state forced vibration tests. The results from these are directly related to the creep and relaxation functions (2, 3, 4).

The results from any one of these tests serve to completely characterize the uniaxial behavior of the material. For example, use of the superposition principle and the creep function J(t) leads to

$$\epsilon(t) = \int_{-\infty}^{t} \frac{d\sigma}{d\tau} J(t - \tau)d\tau \tag{7}$$

which gives the current strain, $\epsilon(t)$, in terms of the history of uniaxial stress, $\sigma(\tau)$. In a similar manner, the relaxation function yields

$$\sigma(t) = \int_{-\infty}^{t} \frac{d\epsilon}{d\tau} G(t - \tau)d\tau \tag{8}$$

Equations 7 and 8 are termed hereditary integrals and can be used to represent the uniaxial stress-strain relationships.

An alternate approach acknowledging the rate dependence of the material is to use a differential operator representation. In this case, the uniaxial stress-strain relationship assumes the form

$$\sum_{n=0}^{N} a_n \frac{d^n}{dt^n}\epsilon(t) = \sum_{m=0}^{M} b_m \frac{d^m}{dt^m}\sigma(t) \tag{9}$$

where a_n and b_m are constants. Sometimes it is found convenient to use rheological models to motivate Eq. 9. To complete the characterization, we must solve Eq. 9 for an appropriate test. Matching this solution with the material response permits the evaluation of the constants a_n and b_m. Clearly the accuracy of this representation depends on N and M in the differential operator representation.

Three-Dimensional Characterization

As indicated earlier, the shear and volume responses of the material are independent of each other provided the material is linear and isotropic. For this reason the strain tensor is divided as

$$\epsilon_{ij} = \epsilon_{ii}\delta_{ij}/3 + \epsilon'_{ij}$$

or

$$\epsilon_{xx} = \frac{1}{3}(\epsilon_{xx} + \epsilon_{yy} + \epsilon_{zz}) + \epsilon'_{xx}$$

$$\epsilon_{xy} = \epsilon'_{xy} \tag{10}$$

In Eq. 10, the first term represents the volume change without shear distortion, while

$$\epsilon'_{ij} = (\epsilon_{ij} - \epsilon_{ii}S_{ij/3})$$

or

$$\epsilon'_{xx} = 2\epsilon_{xx/3} - \frac{1}{3}(\epsilon_{yy} + \epsilon_{zz})$$

$$\epsilon'_{xy} = \epsilon_{xy}$$

represents shear distortion without volume change.

In a similar manner the stress can be decomposed into a hydrostatic state and a deviator state

$$\tau_{ij} = \tau_{ii}S_{ij/3} + \tau'_{ij}$$

or

$$\tau_{xx} = \frac{1}{3}(\tau_{xx} + \tau_{yy} + \tau_{zz}) + \tau'_{xx}$$

$$\tau_{xy} = \tau'_{xy} \tag{11}$$

where $\tau_{ii/3}$ represents the mean hydrostatic stress and

$$\tau'_{ij} = \tau_{ij} - \tau_{ii}S_{ij/3}$$

or

$$\tau'_{xx} = \frac{2}{3}\tau_{xx} - \frac{1}{3}(\tau_{yy} + \tau_{zz})$$

$$\tau'_{xy} = \tau_{xy}$$

is the deviator state with zero hydrostatic pressure.

Because the material is linear and isotropic, the volume change depends only on the mean hydrostatic pressure

$$\epsilon_{ii} = F_1(\tau_{ii/3}, t) \tag{12}$$

whereas the shear distortion is a function only of the deviator stress

$$\epsilon_{ij} = F_2(\tau_{ij}, t) \tag{13}$$

From Eqs. 12 and 13 and the uniaxial characterization, it is clear what tests must be performed and what form the functions F_1, F_2 assume. Equation 12 indicates the need for a creep test in hydrostatic compression with a resulting volume creep function $J_v(t)$ and general time-dependent relationship

$$\epsilon_{ii} = \int_{-\infty}^{t} \frac{d}{d\tau}(\tau_{ii})J_v(t - \tau)d\tau \tag{14}$$

Similarly, Eq. 13 suggests a creep or relaxation test in pure shear. Labeling the shear creep function by $J_s(t)$, we then get

$$\epsilon'_{ij} = \int_{-\infty}^{t} \frac{d}{d\tau}(\tau'_{ij})J_s(t - \tau)d\tau \tag{15}$$

Combining these with Eq. 10 yields

$$\epsilon_{ij} = S_{ij/3}\int_{-\infty}^{t} \frac{d}{d\tau}(\tau_{ii/3})J_v(t - \tau)d\tau + \int_{-\infty}^{t} \frac{d\tau'_{ij}}{d\tau}J_s(t - \tau)d\tau \tag{16}$$

which gives the final desired three-dimensional constitutive equation.

It still remains to verify the validity of superposition in three dimensions. Suitable three-dimensional tests and comparison with the predictions from Eq. 16 serve this purpose as well as indicate its range of applicability for various stress paths.

If desired the shear test indicated may be replaced with the uniaxial tension or compression test mentioned in the uniaxial characterization discussion. Either test in conjunction with the hydrostatic compression test serves to complete the three-dimensional characterization, provided that its range of applicability is verified for three-dimensional stress states.

In the preceding it has been assumed that all tests have been performed at the same temperature under isothermal test conditions. In fact the time-dependent properties of materials are quite temperature-sensitive. This very feature can be used to an advantage in the characterization of the material. Although no mention of this has been given here, there is a well-developed theory and body of knowledge on the temperature-dependence of the rate-dependent material properties (8, 9).

NONLINEAR, RATE-INDEPENDENT MATERIAL

In the following it is assumed that suitable tests have been performed demonstrating that the material is rate-independent and nonlinear.

Deformation Law

A deformation law (5) is simply a device for giving the stress at a point in terms of the total strain at that point, as measured from some reference configuration. This result is independent of the manner in which the total strain is arrived at. The difficulty with such a law is that predictions do not correctly account for any change in the type of loading, i.e., torsion superposed on tension. In the case of proportional loading (defined as loading that produces stress states such that $\tau_{I}:\tau_{II}:\tau_{III} = \alpha:\beta:i$ where τ_{I}, τ_{II}, and τ_{III} are the principal stresses; in addition, principal axes are not permitted to rotate) the deformation law gives correct results. The application and limitations of such a law are discussed by Chang et al. (6).

A general form for a deformation stress-strain relation for an isotropic material under loading is

$$\tau_{ij} = C_1 S_{ij} + C_2\epsilon_{ij} + C_3\epsilon_{ij}\epsilon_{ij}$$

or

$$\tau_{xx} = C_1 + C_2\epsilon_{xx} + C_3(\epsilon_{xx}^2 + \epsilon_{xy}^2 + \epsilon_{xz}^2)$$

$$\tau_{xy} = C_2\epsilon_{xy} + C_3(\epsilon_{xx}\epsilon_{xy} + \epsilon_{xy}\epsilon_{yy} + \epsilon_{xz}\epsilon_{zz}) \tag{17}$$

where C_1, C_2, and C_3 are arbitrary functions of the three strain invariants. The nature of these functions and the values of the material constants contained in them can only be determined by suitable experimentation. An example of this approach is given elsewhere (6). It should be emphasized that this approach is only suitable for proportional loading; otherwise, approximate results are obtained, the degree of approximation being unknown. (It is assumed here that the material is not elastic. In the event that it is elastic, Eq. 17 is correct for loadings that are not proportional as well as for cases where unloading occurs.)

From examination of Eq. 17 it is clear that there is interaction between the shear distortions and hydrostatic state. This feature suggests several useful experiments to determine the material constants, i.e., measurement of the volume change arising from a state of pure shear.

Incremental Law

The first key step in the development of an incremental law (7) is the decomposition of the strain measure into elastic and inelastic components, i.e.,

$$\epsilon_{ij} = \epsilon_{ij}^{e} + \epsilon_{ij}^{p}$$

or

$$\epsilon_{xx} = \epsilon_{xx}^{e} + \epsilon_{xx}^{p} \tag{18a}$$

where ϵ_{ij}^{e} represents the elastic or recoverable strains and ϵ_{ij}^{p} represents the plastic strains that remain upon removal of the load.

It is common, although not necessary, to assume that ϵ_{ij}^{e} is a linear function of the current state of stress

$$\epsilon_{ij}^{e} = \left(\frac{1+\nu}{E}\right)\tau_{ij} - \frac{\nu}{E}\tau_{ii}S_{ij}$$

or

$$\epsilon_{xx}^{e} = \frac{1}{E}\tau_{xx} - \frac{\nu}{E}(\tau_{yy} + \tau_{zz}) \tag{19a}$$

where E and ν are elastic constants. It only remains to describe the relationship between the plastic strains and the stress.

Before this is done, the preceding results are expressed in incremental form. The final constitutive law derived is to only hold for an increment of strain. The total strain is obtained then by integration of the incremental results. If we let $d\epsilon_{ij}$ denote the increment, Eqs. 18a and 19a become

$$d\epsilon_{ij} = d\epsilon_{ij}^{e} + d\epsilon_{ij}^{p}$$

or

$$d\epsilon_{xx} = d\epsilon_{xx}^{e} + d\epsilon_{xx}^{p} \tag{18b}$$

and

$$d\epsilon_{ij}^{e} = \left(\frac{1+\nu}{E}\right)d\tau_{ij} - \frac{\nu}{E}d\tau_{ii}S_{ij}$$

or

$$d\epsilon_{xx}^{e} = \left(\frac{1+\nu}{E}\right)d\tau_{xx} - \frac{\nu}{E}(d\tau_{xx} + d\tau_{yy} + d\tau_{zz}) \tag{19b}$$

The relation of the plastic strain increment to the stress state hinges on the notion of a yield surface. A yield surface, S, is defined by an equation of the form

$$F(\tau_{ij}, K) = F(\tau_{xx}, \tau_{yy}, \ldots, K) = 0 \tag{20}$$

where K is a scalar function of the accumulated plastic strain. If the current stress state is within the yield surface, then additional incremental plastic strains vanish. If the stress state is on the yield surface, then plastic strains will occur if the next increment of stress tends to extend the yield surface.

In this latter case there are a variety of postulates (flow rules) concerning the relation of the plastic strain increment to the stress state and the yield surface. Only one of the simplest is indicated here:

$$d\epsilon_{ij}^{p} = \lambda\tau_{ij}$$

or

$$d\epsilon_{xx}^{p} = \lambda\tau_{xx} \tag{21}$$

The scalar λ depends on the stress-increment and the strain-hardening characteristics of the material.

The final incremental strain-stress equations are given by

$$d\epsilon_{ij} = \left(\frac{1+\nu}{E}\right)d\tau_{ij} - \frac{\nu}{E}d\tau_{ii}S_{ij} \qquad \tau_{ij} \text{ within S} \tag{22a}$$

and

$$d\epsilon_{ij} = \left(\frac{1+\nu}{E}\right)d\tau_{ij} - \frac{\nu}{E}d\tau_{ii}S_{ij} + \lambda\tau_{ij} \quad \tau_{ij} \text{ on S} \tag{22b}$$

The characterization of such a material is difficult. First, the location and character of the yield surface must be experimentally investigated, the strain hardening of the material must be established, and a flow rule must be postulated and verified. The experimental tests are necessarily three-dimensional, and tests to completely characterize the material are extensive in number. Only in the case of certain metallic materials have sufficient experiments been performed.

Note, however, that through the use of the incremental law, yield surface, and flow rule the correct response of the material can be predicted for all types of stress paths and not just for proportional loading.

NONLINEAR, RATE-DEPENDENT MATERIAL

If the material is determined to be nonlinear and rate-dependent, the following constitutive laws might serve as starting points. These constitutive laws are the most sophisticated and general of all the classes discussed. Whatever forms are ultimately selected must be reducible, in some sense, to each of the three previous classifications.

There are two approaches that one may adopt. The first might be considered as a nonlinear generalization of either the linear creep law or the relaxation integral law. An alternate approach is to consider a nonlinear extension of the linear differential operator representation.

Consider first a nonlinear extension of a relaxation integral law—a systematic, time-dependent polynomial expansion of Eq. 3b

$$
\begin{aligned}
\tau_{ij} = {} & S_{ij} \int_{-\infty}^{t} A_1(t-\tau)\epsilon_{ii}(\tau)d\tau + \int_{-\infty}^{t} A_2(t-\tau)\epsilon_{ij}(\tau)d\tau \\
& + S_{ij} \int_{-\infty}^{t}\int_{-\infty}^{t} B_1(t-\tau_1, t-\tau_2)\epsilon_{ii}(\tau_1)\epsilon_{nn}(\tau_2)d\tau_1 d\tau_2 \\
& + S_{ij} \int_{-\infty}^{t}\int_{-\infty}^{t} B_2(t-\tau_1, t-\tau_2)\epsilon_{in}(\tau_1)\epsilon_{in}(\tau_2)d\tau_1 d\tau_2 \\
& + \int_{-\infty}^{t}\int_{-\infty}^{t} B_3(t-\tau_1, t-\tau_2)\left[\epsilon_{ij}(\tau_1)\epsilon_{ii}(\tau_2) + \epsilon_{ij}(\tau_2)\epsilon_{ii}(\tau_1)\right]d\tau_1 d\tau_2 \\
& + \int_{-\infty}^{t}\int_{-\infty}^{t} B_4(t-\tau_1, t-\tau_2)\left[\epsilon_{ii}(\tau_1)\epsilon_{ij}(\tau_2) + \epsilon_{ii}(\tau_2)\epsilon_{ij}(\tau_1)\right]d\tau_1 d\tau_2
\end{aligned}
\tag{23}
$$

Higher order integrals involving powers of strain of order three and greater are neglected in this constitutive law; so it is an approximation only. As seen, Eq. 23 has a total of six kernel functions that must be experimentally determined.

An alternate approach is to examine a nonlinear extension of a differential operator law. This is started by making some assumption concerning the pertinent variables. For example, it might be assumed that the stress is a function of strain, ϵ_{ij}, and strain rate, $\dot{\epsilon}_{ij}$, only; i.e.,

$$\tau_{ij} = F(\epsilon_{ij}, \dot{\epsilon}_{ij})$$

It may be shown (10) that the most general form of F is given by

$$
\begin{aligned}
\tau_{ij} = {} & A_0 S_{ij} + A_1\epsilon_{ij} + A_2\dot{\epsilon}_{ij} \\
& + B_0\epsilon_{ii}\epsilon_{ij} + B_1\dot{\epsilon}_{ii}\dot{\epsilon}_{ij} + B_2(\epsilon_{ii}\dot{\epsilon}_{ij} + \dot{\epsilon}_{ii}\epsilon_{ij}) \\
& + C_0(\epsilon_{ii}\dot{\epsilon}_{in}\dot{\epsilon}_{nj} + \dot{\epsilon}_{ii}\dot{\epsilon}_{in}\epsilon_{nj}) + C_1(\epsilon_{ii}\epsilon_{in}\dot{\epsilon}_{nj} + \dot{\epsilon}_{ii}\epsilon_{in}\epsilon_{nj}) \\
& + D_0(\epsilon_{ii}\epsilon_{in}\dot{\epsilon}_{mn}\dot{\epsilon}_{nj} + \dot{\epsilon}_{ii}\dot{\epsilon}_{in}\epsilon_{mn}\epsilon_{nj})
\end{aligned}
\tag{24}
$$

where A_0, A_1, B_0, B_1, B_2, C_0, C_1, and D_0 are functions of the 10 joint invariants of ϵ_{ij}, $\dot{\epsilon}_{ij}$.

In this case only the linear terms are retained; $A_0 = a\epsilon_{ii} + b\dot{\epsilon}_{ii}$, $A_1 = c$, $A_2 = d$, and $B_0 = B_1 = B_2 = C_0 = C_1 = D_0 = 0$, where a, b, c, and d are constants. Equation 24 then assumes the form

$$\tau_{ij} = (a\epsilon_{ii} + b\dot{\epsilon}_{ii})S_{ij} + c\epsilon_{ij} + d\dot{\epsilon}_{ij} \tag{25}$$

which is the three-dimensional result for a Voigt model. This gives some feeling for the approximation made in the assumption that the stress state is only a function of strain and first strain rate.

If the time dependency is neglected in Eqs. 23 and 24, then both reduce to a deformation law of the type in Eq. 17. Accordingly, these constitutive laws suffer the same defects as a deformation law and are only strictly applicable for cases of proportional loading.

Standard methods for characterizing nonlinear materials have not been established. It is clear, though, that to determine the material functions appearing in Eqs. 23 and 24 it is necessary to run a series of uniaxial and three-dimensional creep and relaxation tests in the nonlinear region. In addition, the region of validity of the proposed law must be experimentally determined.

FRACTURE AND FAILURE

There are a number of possible failure modes of the pavement system. Among these are excessive displacement such as rutting, fracture upon first loading, and progressive cracking under repetitive wheel loads.

The failure analysis is reasonably straightforward in the case of excessive displacements under repetitive loading. The accumulated displacement field is calculated by solving the boundary value problem formulated with the appropriate constitutive law. Assessing the resulting displacement field in light of permissible values then completes the pavement evaluation.

Although fracture of a structure upon first loading is a problem in some fields, it seldom occurs in highway pavements. Accordingly, the failure mode is not discussed further.

Progressive cracking under repetitive loading is a more realistic failure mechanism. To aid in this type of failure analysis, several researchers (12, 13, 14) have examined the fatigue strengths of pavement materials when subjected to cyclic loading. Usually these studies have ignored the substructure of the material and treated the material as a continuum until fracture occurred. Inasmuch as this global approach is reviewed in another paper, the following discussion is restricted to the role the material microstructure plays.

To analyze the effect of the substructure of the material, we must use the principles of modern fracture mechanics (15, 16). The primary hypothesis of this discipline is that all materials possess voids, cracks, or other flaws and that material failures are initiated by the presence of such flaws. The speed and manner in which the voids and cracks grow, propagate, and coalesce determine the type and character of the material fracture. Fracture mechanics is just the field that studies how cracks propagate.

The most fundamental concept in fracture mechanics can be illustrated by considering the problem of a thin, elastic sheet subjected to a uniform tension with magnitude P. Assume that a line crack of length 2a is oriented perpendicular to the direction of tension. The question is, What values of P lead to crack growth?

Griffith (17) used the first law of thermodynamics to derive a fracture criterion for a brittle, elastic material. He reasoned that the unstable propagation of a crack must decrease the total energy of the system. For the crack to start to propagate, the energy released, ΔW_E, must be equal to or greater than the energy needed to create the new crack surface, ΔW_s. For the example problem

$$\Delta W_E = \frac{\partial W}{\partial a}\Delta a = \frac{2\pi P^2}{E} a\,\Delta a$$

$$\Delta W_s = 4\gamma_s\,\Delta a$$

where γ_s is the surface energy associated with the new crack surface. Equating the two gives

$$P_{cr} = \sqrt{\frac{2\gamma_s E}{\pi a}}$$

as the criterion for crack propagation.

Although this simple result is only valid for an elastic and brittle material in a uniform stress field, the basic approach still applies to more complicated problems. In the case of the pavement system the load input arises from the passing wheel load, and, therefore, its time duration is usually short. The materials are time-dependent and nonhomogeneous at the substructure level, further complicating the situation.

Nevertheless, progress is being made on this class of problems. The propagation of cracks in viscoelastic materials is receiving considerable attention at this time. In addition, at least one effort (18) has been made to account for the presence of aggregate in the viscoelastic fracture process. Finally, it is worth mentioning the research work of George (19) who has applied fracture mechanics to soil cements.

It is clear that the continued study of the fracture process using modern fracture mechanics is going to lead to an improved understanding of the problem. Not only will this make it possible to estimate crack growth and subsequent failure, but also it should lead to an understanding of how to make highway materials more resistant to this failure mode.

CONCLUSION

The initial testing of a material is primarily to determine the essential character of its response. Once this is done then, by keeping in mind the final application, one can classify the material response within one of the four categories outlined here.

At this stage an appropriate constitutive law is proposed, thereby motivating certain experiments to characterize the material. It is essential at this stage that work be done within the scope of a constitutive equation; otherwise, the material tests and characterization have no meaning. In addition to a determination of the coefficients or kernal functions in the proposed stress-strain relation, an assessment must be made of the validity and extent of applicability of the constitutive equation for general stress and strain states.

Once the material has been characterized, the resulting stress-strain relations can be used in the stress-deformation analysis and engineering predictions. Assessing the accuracy of these predictions requires that reference be made to the accuracy of the material characterization and its region of applicability.

In the event that the stress and deformation fields used in the failure analysis do not depart from the region of applicability of the stress-strain relation, the failure analysis proceeds on a straightforward basis. However, if the material performance deviates from its model, such as by cracking, this must be properly accounted for. In particular the material characterization involves, then, not only determining the stress-strain relations but also developing the failure or fracture law that limits the range of applicability of the material.

Finally, it should be pointed out that no effort has been made in this paper to present a complete and rigorous treatment of constitutive equation theory. However, it is hoped that this paper might indicate the scope, capability, and current state of material characterization. Meaningful stress analyses and engineering predictions hinge on suitable material representation, and this must necessarily be done within the scope of a proper three-dimensional constitutive equation and appropriate failure criterion.

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SOLUTIONS AND SOLUTION TECHNIQUES FOR BOUNDARY VALUE PROBLEMS

Keshavan Nair

One of the practical objectives of the theories of mechanics is to assist in the solution of engineering design problems by providing the theoretical basis for determining the response of a system to a variety of prescribed inputs. This is done through the formulation and solution of boundary value problems. There is one major factor that has, during the last decade, changed the entire approach to the solution of boundary value problems. This is the development of numerical techniques that, in conjunction with the availability of high-speed digital computers, permit the solution of complex boundary value problems. The availability of operational computer programs has made it possible for the average practicing pavement engineer to conduct analyses that only a few years ago would have been considered impractical. One of the major thrusts in achieving progress in pavement design is the use of this capability.

In writing this paper, I find there is the dilemma of whether to strive for completeness in the theoretical aspects or to try to answer in broad terms the two questions of what is the present state of the art in this area and of where the future effort should be. This paper takes the broad approach. It is felt that those whose interest is in the detailed theoretical aspects are sufficiently familiar with recent developments and that a relatively brief summary is not likely to provide them with new information. Furthermore, inclusion of a detailed theoretical discussion is likely to make the paper less readable to those whose major interest is application. The paper attempts to point out those solutions and solution techniques available at the present time in a form that can be used in design. Because one of the major objectives of the workshop is to look toward the future and encourage an exchange of ideas with regard to future research and research in progress, this paper also discusses directions of future research and development. However, before proceeding to a discussion of various methods of solution, I should comment briefly on the formulation of a boundary value problem.

FORMULATION OF THE PROBLEM

The formulation of a boundary value problem involves idealizing the real physical problem and casting it into mathematical form. For the class of problems representative of pavement systems, the mathematical form of the boundary value problem is a set of partial differential equations subject to various initial and boundary conditions.

There are three essential components to a boundary value problem: (a) governing equations, (b) constitutive equations, and (c) boundary and initial conditions. For the analysis of pavements, the governing equations are the equations of equilibrium, motion (for dynamic problems), and compatibility. These equations are derived from the basic laws of classical physics and from continuity considerations in the material. Various approximations can be introduced at this level (e.g., small strains to obtain linearity and symmetry of the stress tensor). It should be recognized that the governing equations are independent of any material properties.

Constitutive equations are representations of the properties of the particular materials under consideration and represent idealizations of actual material behavior.

Boundary conditions may consist of prescribed displacements and stresses on various boundaries. (For thermal and hygro stresses it is necessary to define the temperature and moisture contents as functions of space and time.) For static problems this is sufficient; for dynamic problems it is necessary to specify the conditions at some arbitrary time, generally at $t = 0$, when they are called initial conditions. The governing and constitutive equations can only be solved in general terms; it is boundary and initial conditions that make the general solution specific for the problem under consideration. The boundary and initial conditions also represent various levels of idealization. For example, the actual time variation of load might be approximated by a simple analytic function (e.g., sine), or nonaxisymmetric loads might be approximated by axisymmetric load distributions.

It is appropriate to comment briefly on how these three components are accounted for in modeling a pavement section. The governing equations are of general applicability. The materials composing the various layers have to be represented by appropriate constitutive equations. This aspect is discussed in a separate paper. (See the Appendix for bibliography.) In addition, the loading conditions and the geometry of the problem have to be approximated in order to make the problem amenable to solution.

The loads are dynamic and can be considered to be applied randomly with regard to both space and time. All current methods of pavement analysis and design treat the loads deterministically. The other basic assumption on loading conditions is whether to treat the loads as static or dynamic. All design methods in use at the present time treat the load as static. Analytical solutions for dynamic problems have been developed on the assumption that inertial effects can be neglected. Analyses have shown that for highway traffic this is a reasonable assumption. These solutions are only of interest when the materials in the pavement system are rate-dependent.

In practice, multiple loads are applied to the pavement. For linear problems the fundamental problem is based on the application of the single load; the multiple load situation can be treated by superposition of single load solutions. The shape of the loaded area and the distribution of the load over this area depend primarily on the tire, the inflation pressure, and the characteristics of the surface layer. All current design methods using analytical solutions make the assumption that the load is distributed uniformly and can be considered axisymmetric. If the physical geometry of the problem is axisymmetric, then nonaxisymmetric load distributions can be analyzed for linear problems.

Assumptions regarding the geometry of the total structural problem depend primarily on the location of the loads relative to the edges of the pavement. This leads to the two assumptions that have been used in the structural analysis of pavements: the layered system theory and the thin plate theory. If the loads are sufficiently distant from the edges of the pavement, in that the effects of the boundary on the stresses in pavement can be neglected, then the problem can be treated as a layered system in which each layer is of infinite horizontal extent. When edge effects are important the extent of the pavement layer has to be considered finite in extent, and the thin plate theory is used because it is not possible to handle the general three-dimensional problem. This latter approach has been used almost exclusively for the design of concrete pavements.

The increased availability of computer programs, which utilize various numerical techniques to solve boundary value problems, has resulted in a tendency to decrease the attention paid to the formulation of the problem. The importance of a correctly formulated boundary value problem cannot be too strongly emphasized. No solution technique, irrespective of the degree of sophistication, can provide adequate answers for design if the problem has been formulated incorrectly.

METHODS OF SOLUTION

There are two basic techniques for obtaining solutions to boundary value problems. These are analytical (sometimes referred to as classical) and numerical. Because the final objective in the case of a practical problem is a numerical result, no solution relies entirely on one of these techniques. A numerical solution technique will use a problem formulation that will be directed toward a computational procedure from the outset, whereas an analytical technique will carry the solution as far as possible before resorting to numerical calculations.

Numerical techniques require that various approximations be made in developing the solution to boundary value problems. Because of the increased availability of "ready-made operational" computer programs, there is a tendency to ignore the effects of possible errors from making discrete and rounding off and questions of instabilities and convergence. Whereas the available programs are satisfactory in the solution of most linear elastostatic problems, these effects must be considered in the analysis of nonlinear and dynamic problems. Possible errors and questions of stability and convergence are discussed briefly in terms of the finite difference and finite element techniques.

As pointed out earlier, the formulation of a boundary value problem is in terms of differential equations. In the finite difference approach, the basic principle is that the derivative can be represented in discrete form. The differential equations are then represented by difference equations. A difference equation approximation must satisfy the requirement that as the mesh size goes to zero the differential equation is obtained. Furthermore, as the mesh size decreases the numerical solution should approach the "exact" solution. In a numerical solution there are errors due to discretization that are dependent on the mesh size and errors due to rounding off that occur because of the truncation of numbers in a computer. An important consideration in numerical techniques is stability; this applies specifically to step-by-step procedures. Because there is some error with each step, the computational scheme should be such that the error does not grow too rapidly.

In the finite element technique, the physical problem is approximated by dividing the solid body into a series of elements. This method, like the finite difference method, produces solutions that have discretization and rounding-off errors. A balance must be obtained between the need for accuracy, which requires a large number of elements, and the need for minimizing computer time, which increases with an increase in the number of elements. For dynamic and nonlinear problems, where step-by-step procedures are used, stability considerations are important.

When available solutions and solution techniques are examined, it is possible to subdivide them into a variety of categories. Because of the emphasis of this paper it is appropriate to consider them in the following two categories: (a) solutions and techniques for linear problems, and (b) solutions and techniques for nonlinear problems. Although this division may appear artificial in that techniques for solving nonlinear problems are generally applicable to linear problems, the subdivision is particularly appropriate when past achievements and future goals are considered.

SOLUTIONS AND SOLUTION TECHNIQUES FOR LINEAR PROBLEMS

Elastic Layered Systems

Analytical Solutions—The geometrical domain of a layered system, i.e., the semi-infinite domain, and the regularity or "at-rest" conditions that exist at the boundaries make the problem particularly suited to analytical treatment. To obtain a solution to a layered system problem requires that the boundary condition be satisfied as well as the continuity conditions between the various layers. In principle, once the two-layer problem is solved, the methodology for solving the general multiple-layer system problem is established. It should be recognized that each layer is considered homogeneous and isotropic.

Since the original work by Burmister, the layered system problem has been analyzed extensively with a view toward obtaining numerical results and recasting the solution into a more general form. Satisfying the continuity conditions at the interface requires the solution of a number of algebraic equations, which include evaluation of infinite integrals. For most problems, involving more than two layers, the only practical method for obtaining a solution is to use a digital computer for the solution of the algebraic equations and the evaluation of the integrals. Practicing engineers should note that computer programs are now available for solving layered system problems formulated in accordance with the methodology first outlined by Burmister. The number of layers these programs can handle covers the range of all practical problems. Because these programs have been "debugged" and are operational, their use in routine design is a practical proposition. These programs provide information on stresses, strains, and displacements throughout the pavement system.

Numerical Solutions—There are fundamentally two numerical techniques currently in use for the solution of boundary value problems that are representative of the pavement system: the finite difference technique and the finite element technique. Of these two techniques, the latter appears to have the greatest potential. Details of the method have been discussed in the literature (see Appendix). The development of the finite element technique is directed toward a computational method, and the objective, from a practical standpoint, is to obtain a computer program that can efficiently solve pertinent boundary value problems. At the present time there are available, for general use, operational computer programs that can be used to solve anisotropic and nonhomogeneous linear elastostatic problems under axisymmetric conditions. It should be noted that, in comparison with the layered system formulation, the finite element technique is not subject to the restriction that each layer be homogeneous and isotropic. This property is of significance when it is necessary to include the effect of stress level on material properties. The cost of doing such analyses is minimal in terms of computer time, and the output provides information on stresses, strains, and displacements throughout the pavement section. Because of the rapidity with which these problems can be solved, it is relatively simple for the designer to study different designs.

In the analysis of thermal problems, it is assumed that the distribution of temperature can be obtained by solving the diffusion equation independently. This implies a decoupling of the temperature and stress effects. Finite element and finite difference programs are available for solving the diffusion equation to obtain the temperature distribution. Once the temperature distribution is obtained, its influence both in the form of a change in material properties or in introducing thermal stresses can be readily accounted for. Available finite element computer programs for the analysis of axisymmetric solids have the capability to analyze these temperature effects.

If inertial effects are neglected, the solutions to the moving load problem can be obtained by superposition of static solutions. Although it has been shown that inertial effects can be neglected at conventional highway traffic speed, it should be recognized that the finite element technique provides a means for analyzing dynamic problems and that operational computer programs are available.

Although computer programs for the axisymmetric case are generally available and are operational, it should be recognized that the actual problem belongs to the general three-dimensional class. Programs for solving such problems have been developed; however, they are not available for general use at the present time. The cost for conducting a three-dimensional analysis is far greater than that for an axisymmetric analysis. A possible technique for overcoming this disadvantage is to treat the problem in a general three-dimensional sense in the area in the close vicinity of the load and in an axisymmetric manner at a sufficient distance away from the load. Another possible alternative is to invoke St. Venant's principle and, instead of at-rest boundaries, to use the results from closed-form solutions of simple loading conditions at boundaries that can be located at much smaller distances from the loaded area than the at-rest boundaries. This would decrease the number of elements required to model the problem, reducing the computer time.

Thin Elastic Plate Formulation

As pointed out, the thin plate formulation is primarily used to consider the effect of edge loading conditions. The formulation has been used almost exclusively for the design of concrete pavements. It is a two-layer system, i.e., a plate and a supporting medium.

Analytical Solutions—The fundamental problem of concentrated load acting on a thin elastic plate resting on a foundation was solved by using the integral transform approach by Holl. Since that time numerous solutions have been developed for plates on various types of supporting media. The use of the thin plate theory to the design of pavements was first proposed by Westergaard; modifications based on theoretical and experimental considerations have been proposed by numerous investigators. Available analytical solutions for use in practice, do not consider nonhomogeneous and anisotropic material properties and do not adequately account for joints, cracks, and possible loss of support.

Numerical Solutions—The finite difference technique has been used fairly extensively in the analysis of plate problems. However, because of the difficulties in handling corners and because of the physically motivated formulation of the finite element method, most of the new developments in the analysis of plate problems are likely to be with the use of finite element techniques. Currently available operational computer programs can analyze plates resting on an elastic foundation under fairly general conditions. These include orthotropy nonhomogeniety in the soil and plate and localized loss of support.

Viscoelastic Layered Systems

As in elasticity, the boundary value problem consists of solving the governing differential equations subject to the appropriate boundary and initial conditions that represent the physical problem to be solved. The boundary conditions may be in the form of prescribed boundary stresses and displacements with regard to time and position, and the initial conditions indicate the conditions at time $t = 0$.

Because time occurs in both governing equations and boundary conditions, it is possible that the boundaries of prescribed stress and displacement may vary during the loading history. Such situations arise in the Hertz problem where with time the sphere indents the material, resulting in a change of the area of contact, thus varying the areas of prescribed displacement and stress. The total volume or surface area of the material may also change with time. Such situations occur in the case of crack propagation and in the case of ablating rocket propellents.

For pavement problems it is assumed that variations of surface and volume do not occur and that areas of prescribed deflection and stresses do not vary during the loading process. The formulation of the boundary value problems in viscoelastic systems is identical to that in elastic systems with the exception that stress-strain laws are time-dependent.

Analytical Solutions—Except for the simplest problems, solutions to boundary value problems in viscoelasticity rely on the extensive use of digital computers. The early emphasis in analyzing problems used the Laplace (or Fourier) transform technique, recognizing that a viscoelastic problem could be compared to some equivalent elastic problem in the transformed domain. The elastic solution has then to be inverted to obtain the required time-dependent solutions. This inversion can be extremely difficult and is only practical for simple representations of viscoelastic response in the form of differential operators. For realistic representations of viscoelastic response, it is necessary to use integral representations of the stress-strain-time relation of the materials based on experimentally measured creep or relaxation functions. In this case the Laplace transform technique leads to considerable difficulties, and it is more convenient to proceed directly with a spatial transform. The satisfaction of the continuity conditions at the interface leads to a set of simultaneous integral equations. Numerical solution of these equations and numerical evaluation of the spatial inversion result in the required solution. By using these techniques, we can apply experimentally obtained curves directly in the calculation. Solutions to the moving load problem, neglecting inertial effects, and a load applied periodically have been obtained by superposition of the static load solutions. The influence of the velocity and the period of application are of significance because of the time-dependent character of viscoelastic materials. Available solutions permit the computation of cumulative deformations as a function of load application and time. All currently available solutions are for the axisymmetric case.

For the practicing engineer it is necessary that operational computer programs be available for use in design. A number of computer programs using this approach have been developed. However, the dissemination of the information has been slow. It is necessary to present this information in a manner that will encourage its use by practicing engineers. This has not been done.

Numerical Solutions—The finite element technique has been applied to the solution of linear viscoelastic problems. Operational computer programs are now available, though their use has not yet become prevalent. It should be recognized that the computer time involved is far greater for solving viscoelastic problems than for solving elastic problems.

SOLUTIONS AND SOLUTION TECHNIQUES FOR NONLINEAR PROBLEMS

Although the importance of nonlinear analysis in the context of the total structural design of a pavement system has not been established, nonlinear problems must be solved if more realistic constitutive equations are to be used. It should be recognized that nonlinearity can also be introduced by large displacements, i.e., kinematic nonlinearity. Obtaining solutions to nonlinear problems depends almost exclusively on numerical techniques.

A number of ad hoc modifications of linear elastic theory have been used to solve "more realistic" boundary value problems. These modifications include a dependence of the modulus in a linear elastic analysis on stress. The problem is solved by iteration until a solution within some specified degree of convergence is obtained. Computer programs using the finite element technique to solve these ad hoc nonlinear problems are available, and their use does not present any more difficulty than does the use of linear programs.

It would appear at the present time that the finite element technique has the greatest potential for solving nonlinear problems. For illustrative purposes, consider the case of a nonlinear elastic constitutive law under the assumption of small strains.

Two methods of analysis commonly used for nonlinear problems are the incremental method and the Newton method with constant or variable slope.

The incremental load method breaks up the applied loads into n increments; a linear elastic solution is then sought for each increment, and the final solution is the sum of the increments. This procedure can use an incremental representation of the nonlinear elastic law along with a standard finite element computer algorithm for the solution of axisymmetric linear elastic boundary value problems (or any other method of solving linear boundary value problems). As an example, consider a layered system subjected to a uniformly loaded circular area. The pressure is divided into n increments and applied incrementally. The response of the system to the first increment can be obtained from the usual linear elastic theory. This will make possible the determination of principal strains throughout the layered systems.

From these known principal strain states the incremental moduli S_{ij} can be obtained for the next linear problem arising from the application of the second increment of applied pressure. The same procedure is repeated until the total load is applied. In terms of finite element nomenclature, the procedure requires that a new stiffness matrix be calculated for each load increment. By examining the range of principal strain states (both in magnitude and in ratios of components), we will be able to select the types and numbers of experiments required to characterize the nonlinear material behavior. This will in general vary with the particular problem (loading, layer thickness, or layer mechanical properties) and will require considerable cooperation between analysis and experimentation, perhaps ultimately linking the two into an integrated device for performing characterization and analysis.

The incremental approach can be used for conducting an elastoplastic analysis. It will be necessary to include a yield criteria to determine whether the material is in the plastic range. In addition, it will be mandatory to keep track of the stress path to determine whether loading or unloading is occurring. The same principles of incremental characterization and solution can be used for nonlinear viscoelastic solids. However, both the characterization of materials and the stress analysis algorithm are considerably more time-consuming.

The constant slope method places the nonlinear portion of the stiffness on the right side of the governing equation as a forcing function. The stiffness (slope) is the same for all iterations. The solution is then obtained by iteration. The variable slope method differs only in that the slope is updated after each iteration.

Finite element analyses based on the theorem of minimum potential energy are known to yield very accurate solutions for the displacements but frequently yield poor results for stresses. The few nonlinear solutions that have been attempted by using this method have not behaved well when the nonlinearity exceeds about 10 percent. However, the Hellinger-Reissner variational theorem provides a solution to these difficulties. Because both the displacements and stresses are included as primary variables, the resulting stresses are of a much improved accuracy and do not possess the spatial oscillations often found in displacement methods. The governing equations become the stress equations of equilibrium and the stress-strain law; whereas in the displacement formulation, the governing equations are the displacement equations of equilibrium. This is significant because the stress equations of equilibrium often are independent of the material properties, even for physically nonlinear materials. Thus, it is possible to obtain accurate solutions by using Newton's constant slope method with the above-mentioned method of analysis in a very few iterations. Displacement methods generally require many iterations that use more sophisticated models and that still fail at moderately high nonlinearities. Research done in this area indicates that the constant slope method has been found to be superior to the incremental load method when the mixed model is used and sufficiently convergent to give accurate results in a few iterations.

Another solution technique that may be of considerable significance in developing solutions for nonlinear boundary value problems is the technique of quasi-linearization developed by Bellman and his colleagues. The technique has been applied primarily to the solution of ordinary differential equations. In this context it is of considerable significance to problems in system identification, which are an important aspect of material characterization. The application of quasi-linearization to nonlinear partial differential equations, though limited, does indicate that it might be a powerful tool for the solution of such problems. It is claimed that the method has very good convergence and stability characteristics.

It is extremely important to recognize the problems associated with convergence when solution techniques are developed and used for nonlinear problems. It is not possible to check all facets of a computer program by comparison with known analytical results. Therefore, the use of solution techniques for nonlinear problems requires experience and judgment. It is important to realize the interdependence of the material characterization and solution techniques. If progress is to be made in the area of nonlinear analysis, research in these two areas will have to be closely coordinated. A great deal of work is needed in developing solutions for nonlinear problems before they can be used in practical designs.

SOLUTION TECHNIQUES—THE GENERAL PROBLEM

If, as is currently being advocated, a systems approach to the design of pavements is used, then it will be necessary to consider solution techniques for aspects of the problem other than the structural aspect. In this brief paper one can only call attention to certain tools that should be considered in an analysis of the pavement problem.

Dynamic programming in its application to optimization problems will be of considerable significance if optimum designs are to be developed. This will include the application of quasi-linearization techniques. One approach to the optimization of a design of the pavement system is to treat the system as an adaptive control process. In such processes we consider the problem of optimizing a process where our knowledge increases during the process. Dynamic programming is particularly suited to handling such problems. It should be recognized that dynamic programming is readily applicable to stochastic problems.

A great deal of work needs to be done in the application of these techniques to the pavement design problem.

FINAL REMARKS

In assessing where we are and where we are going with regard to solutions for boundary value problems, we should consider practical application and research separately. For routine structural design, operational computer programs for analyzing the axisymmetric linear elastostatic problem under very general conditions are available. The stresses, strains, and deflections in pavement sections can be determined and, in conjunction with various theories on modes of failure, e.g., fatigue, can be used in practical design. Because of the speed with which calculations can be performed, the designer can examine many combinations of materials and thicknesses. It can be concluded that the researcher has finished his task in the area of axisymmetric linear elastostatic problems and that the burden is on the designer to use this information.

In the area of linear viscoelasticity, sufficient theoretical work for axisymmetric linear analysis appears to have been completed for use in design. It remains now to disseminate the information to the practicing engineer and to provide guidance in its use.

From the research standpoint, there are a number of areas where further work appears necessary.

1. Conduct general three-dimensional linear elastic analysis. At the present time finite element computer programs are available to conduct such analyses. However, these programs cannot be considered operational in the sense that they can be used in routine design. Certain modifications will be necessary to tailor the programs for the efficient analysis of pavement systems.
2. Develop solution techniques and solutions for nonlinear problems. Considerable progress in this area has been accomplished; however, there has been little application to the pavement problem. The work to be done in the area cannot be done independently of nonlinear material characterization techniques.
3. Include stochastic and probabalistic concepts in analysis. The variability of materials and the nature of loads lead to the conclusion that complete analyses will require the inclusion of stochastic and probabalistic considerations in the solution of boundary value problems.
4. Develop solution techniques for the total pavement system problem. In considering the total pavement problem, researchers have spent a great deal of effort in developing various diagrammatic models of the pavement system. However, very little quantitative work has been done. It is now time to devote some effort in this direction. A possible avenue of future research is in the application of dynamic programming to the optimization of a pavement design system.

Before proceeding with research in various specific areas, we should direct the first effort toward integrating available solutions for boundary value problems with the other available information necessary for the development of a working structural design system and toward making it available for general use. There has been more progress in research than is generally realized. Once such a working design system is established, research needs for structural design can be better defined by sensitivity studies.

A major objective of engineering research is practical application. Therefore, it is imperative that the profession examine its needs for research in terms of significance in the context of the total system. For example, does it matter if materials are characterized as linear or nonlinear in the context of the variability in materials due to construction techniques? Has the structural design reached a degree of refinement that is sufficient in terms of the design of the total system? Perhaps this is a time for consolidation and integration of past scattered efforts and for evaluation of what is needed. However, we should not fall into the trap of trying to fit the needs to our capabilities. Rather we should expand our capabilities to fit the needs.

APPENDIX

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Editor's Note: The original paper included an extensive annotated bibliography, which has not been printed here. This bibliography is available in Xerox form from the Highway Research Board at cost of reproduction and handling. When ordering, refer to XS-38, HRB Special Report 126.

DAMAGE AND DISTRESS IN HIGHWAY PAVEMENTS

Fred Moavenzadeh

An understanding of what constitutes failure in pavement structures and of the factors that contribute to damage initiation, propagation, and accumulation is important to the development of a rational method of design. This study reviews the state of the art in the area of pavement damage with a view to identifying the pertinent damage mechanisms.

The performance of a pavement structure in a given traffic and climatic environment may be defined as its ability to provide an acceptable level of serviceability with a specified degree of reliability for an assumed level of maintainability (1). The impairment or loss in the ability to provide the necessary services in a given locale may then be considered as the "failure" of the pavement. When viewed in this context, failure becomes a loss in performance; it is the extent to which the pavement has failed to render itself serviceable, and it results from an accumulation of damage during a given time period. The failure age or the life of the pavement is, then, the time during which serviceability deteriorates to an unacceptable level as determined by the users.

The question of what is the unacceptable level of serviceability at which the pavement must be considered failed is a highly subjective one. It depends on the user's evaluation of the performance, and it involves many intangible and not easily quantifiable factors such as cost, comfort, convenience, and safety. The problem is further compounded by the fact that there does not exist a usually accepted and comprehensive function describing the pavement serviceability in terms of damage parameters. At present one can only assume that pavement failure is a many-sided problem, and it is the result of a series of interacting complex processes, one of which is clearly understood.

Therefore, an integrated comprehensive picture of failure is obtained by studying in detail each of its components. From such studies, methods may be developed for analyzing the effect that each component has on the behavior of the structure. Finally, all of those methods may be grouped together in a meaningful way so that, for any given environment, the performance history of a pavement can be predicted. This study investigates failure from the "structural integrity" viewpoint and places special emphasis on load-associated factors affecting the structural integrity. It must, therefore, be emphasized that this is only one aspect of the failure problem.

THE STRUCTURAL INTEGRITY OF THE PAVEMENT

The structural integrity of a pavement may be defined as its ability to resist destruction and functional impairment in a particular traffic and climatic environment. Under the combined destructive action of these elements, several distress mechanisms develop within the structure and propagate either independently of each other or through interacting complex processes to produce eventually any or all of these broad groups of distress: disintegration, distortion, and fracture (rupture). To develop a thorough understanding of these mechanisms, one has to trace the path from the time of initiation, through propagation, to that of global manifestation.

Although it is realized that field behavior results from a series of interacting complex processes and that all distress mechanisms must be analyzed in order to present not only a realistic but also a totally comprehensive picture of damage behavior, only the distress occurring as a consequence of mechanical loading has been investigated. The motivation for doing this stems from the fact that an extensive amount of work has been done experimentally on the modes of damage associated with mechanical loading.

This makes it possible, to a certain extent, to adopt certain assumptions as to the manner of damage propagation in this mode of loading.

This study is presented in five broad sections. The discussion begins with a close study of the conditions governing the initiation, propagation, and attainment of critical size of the defect area in engineering materials under arbitrary loading histories. It provides the background necessary for the development of the cumulative damage model.

After the conditions governing the physical failure of engineering materials in general are established, the pavement system is investigated with a view to identifying the variables that affect its performance and that subsequently bring about ultimate distress in a given environment. On the basis of the conclusions arrived at, a framework for cumulative damage is developed.

FAILURE OF ENGINEERING MATERIALS

The intensive interest that has developed during the past several years concerning the accumulation of damage in engineering materials and structures has its roots in the following problems:

1. The life prediction of an engineering material or structure under an arbitrary load history in a given environment,
2. The amount and distribution of damage in the material or structure under the arbitrary loading spectrum mentioned previously, and
3. The manner and rate of accumulation of damage.

This section presents a concept of damage by examining the processes of fracture and flow in solid materials. It describes what the damage is, how it manifests itself, and which parameters can be employed to describe it.

Several observations are made about the distribution and propagation of damage within a material that is under an arbitrary loading history. Some well-known theories and criteria that have been postulated for the failure of engineering materials are discussed. The damage of materials in a repeated loading environment is closely examined.

Concept of Damage

Damage may be defined as a structure-sensitive property of all solid materials; structure sensitivity is imparted to it through the influence of defects in the form of microscopic and macroscopic cracks, dislocations, and voids that may have been artificially or naturally introduced into the material, thereby rendering it inhomogeneous. Characteristically, structure-sensitive phenomena involve processes that grow gradually and accelerate rapidly once an internal irregularity or defect size exceeds a certain limit. Damage may, therefore, be said to occur in a similar fashion.

The progression of damage in an engineering material or engineering structure may occur under the application of uniaxial or multiaxial stationary or repeated loads. The damage progression has been categorized by two different conditions: ductile and brittle. The ductile condition is operative if a material has undergone considerable plastic deformation or flow before rupture. The brittle condition, on the other hand, occurs if localized stress and energy concentrations cause a separation of atomic bonds before the occurrence of any appreciable plastic flow. Note here that no mention is made of a ductile or a brittle material per se. According to von Karman (2), this implies that failure is not in itself a single physical phenomenon but rather a condition brought about by several different processes that may lead to the disintegration of a body by the action of mechanical forces. Damage may, therefore, progress within a material under the different mechanisms of fracture and flow depending on the environmental stress, strain, and temperature conditions. For instance, low carbon steel exhibits fibrous and shear types of fractures at room temperature; below -80 C brittle fracture occurs, and intergranular creep fracture is dominant in slow straining at 600 C and above (3). A material may, therefore, have several characteristic strength values when several fracture mechanisms operate at different critical levels of the stress or strain components.

Although the mechanisms of damage initiation and propagation in the two failure modes are different, they have three major points in common:

1. A particular combination of stress or strain concentration is required to create a defect nucleus;
2. A different combination of stress or strain quantities is then required for the propagation of the defect; and
3. A critical combination of stress and strain concentrations is required for the transition from relatively slow to fast propagation to catastrophic failure.

In addition, the distribution and progression of damage in solid material are a random process that is both spatial and temporal (4, 5, 6, 7), and damage in an engineering material is a statistical process brought about by the interaction of several complex mechanisms. An engineering material or structure can, therefore, fail under a given system of external loadings when either of the following two criteria is satisfied:

1. The distribution of internal flaws is such that excessive deformation is attained (usually for ductile behavior), or
2. The distribution is such that a fracture threshold is reached under an arbitrary loading history (usually for brittle behavior).

Accordingly, during the years several reasons have been advanced as explanations for the observed behavior of "damage" in an engineering material, and based on such explanations several theories have emerged. Researchers have approached the problem both deterministically and statistically from the molecular and phenomenological levels. On the molecular basis, the differences between the fracture mechanisms involved are emphasized because, at this level, the material is essentially discontinuous.

On the macro level, the criteria for fracture are basically similar and utilize the concepts of continuum mechanics. The fracture laws are generally based on either local or global energy, stress or strain concentrations within the material.

Theories like the Eyring rate process (10) developed for viscous materials, the Gnauss theory (11) for viscoelastic materials, and Weibull's theory (12) for brittle materials have attempted to explain on the basis of a statistical model some of the phenomena observed when materials like metals, textiles, concrete, and others fracture under applied stress. The basic assumption is that an assembly of unit damage processes grows in a probabilistic way to yield an observed macroscopic effect, with temperature fluctuations and activation energy distribution playing a significant role. Such statistical theories have a significant advantage over deterministic concepts because they account for the role of chance in the behavior of materials.

Nadai (15) has concluded that, whereas a particular failure theory may work well for a particular class of materials, it may often fail quite hopelessly to predict conditions of failure for another class. Examples of this are evident in the use of the maximum shear stress theory, distortional theory, and octahedral shear stress theory, all of which work very well for metals and can be justified on an atomic scale because of the mode of crystal slip in a polycrystal. However, their applications to the failure of materials such as sand, gravel, and clay are questionable because the shear stress necessary for slip in such materials depends also on hydrostatic pressure. Coulomb treated the failure of these materials as a simple frictional resistance that is proportional to pressure and developed the Mohr-Coulomb theory (16), which has met with reasonable success in soil mechanics. Although this criterion neglects the influence of the intermediate principal stress on failure, Bishop (17) and others have deemed it a satisfactory first approximation for three-dimensional situations as well.

Parameters of Damage in the Repeated Loading Mode

In many materials, the initiation, progression, and ultimate manifestation of distress in the form of fracturing under a repeated load occurs under the action of two separate processes, crack initiation and crack growth, both of which are governed by different criteria. In metals, this behavior has been attributed to localized slip and plastic deformation (18) and to the cyclic motion of dislocations. In polymers and asphaltic mixtures, the cracks initiate from air holes, inhomogeneities, and probably molecular chain orientations and molecular density distributions (20).

The mechanism of crack propagation has been explained by many researchers from a consideration of the energy balance at the crack tip, which deforms as cycling progresses. The propagation is slow when a considerable amount of plastic deformation occurs at the crack tip, which as a result of this becomes blunted. It is fast when the released portion of the stored energy exceeds the energy demand for creating new surfaces.

Erickson and Work (21) discovered that the history of load application had a significant influence on the progression of damage. When the application was a high prestress followed by a low stress, the degree of damage created was greater than when the application was vice versa. The authors explained this occurrence by suggesting that, on the first few cycles of load application, a certain number and distribution of crack sites form depending on the stress level. and the application of subsequent loads merely causes propagation from these sites. The literature contains several phenomenological and molecular theories that handle the problem of life prediction for any material in a given repeated load environment.

Molecular Theories—In some of the molecular theories, statistical mechanics principles and the kinetic reaction rates concept have been utilized. Coleman (24) and Machlin (25) employed the Eyring rate process theory to study the fatigue characteristics of nylon fibers and metals respectively.

Coleman's theory implies that for every material a constant strain level exists at which fracture will occur, but a variety of experimental results shows that this is not the case. Moreover, it does not account for progressive internal damage, inasmuch as only failure conditions are represented. Mott (26) and Orowan (3) have presented for metals fatigue theories that take into account the fact that plastic deformation and strain-hardening occur during fatigue. Mott's theory attributes the formation of microcracks to the occurrence of dislocation within the material. Orowan's theory assumes the presence of a plastic zone within which a crack forms and propagates. Both of these theories have given good agreement with experimental results at times. The discrepancy observed is mainly due to the fact that damage is a stochastic phenomenon, whereas the theories are deterministic in nature. To increase their accuracy requires a statistical approach.

Phenomenological Theories—Despite the fact that several molecular mechanisms have been shown to be operative during fatigue growth in a material, one suspects that the process itself may not be that fundamental in nature. Therefore, instead of searching for molecular theories, we can possibly get a coherent picture from the continuum mechanics approach (with certain reservations).

This has been the motivation behind several phenomenological theories of cumulative damage: Miner's (27), Corten and Dolan's (22), and Valluri's (28) to name a few. The underlying concept in these theories can be illustrated by the work of Newmark (29).

In this approach, it is assumed that when a material is in a given load and climatic environment the degree or percentage of internal damage, D_i, is at any time commensurate with the appropriate number of load repetitions, N_i (i.e., for $0 \leq D_i \leq 1$, $0 \leq N_i \leq N_f$). With this assumption, a damage curve exists for every constant stress or strain repeated mode of loading. Because damage in effect implies that a loss in original capacity can result from either the creation and growth of plastic zones or the initiation and propagation of cracks, the strain developed in the material under load, the crack length, and the rate of crack growth can all be used as damage determinants. This reasoning is currently used in constant amplitude stress or strain fatigue tests.

Generally speaking, the combined effects of damage and recovery processes resulting from microstructural changes imply that the damage curve should have different forms for different stress levels and loading histories. Some damage theories, such as those of Miner and Williams, assume a unique degree of damage caused by a stress cycle ratio (n/N) applied at any time. Williams' theory (30) makes a similar assumption but with respect to time ratios (t/T_f), where t is elapsed time from start of experiment, and T_f is time to failure. Both theories result in a linear summation of the ratios, and both have a major shortcoming in that prior history and sequence of events cannot be accounted for. Despite this shortcoming, Miner's theory has been successful when applied to rate-insensitive materials. Williams' theory had similar success when used for rate-sensitive materials. In Corten and Dolan's theory (22), the damaging effect under a stress cycle is considered dependent on the state of damage at any instant, and the expression for damage is

$$D_t = mrN^a \tag{1}$$

where

N = number of cycles;
r = coefficient of damage propagation, which is a function of stress level;
a = damage rate at a given stress level, which increases with the number of cycles; and
m = number of damage nuclei.

The Corten and Dolan approach is a rational attempt to modify Miner's theory. The determination of the significant parameters m and r, however, requires the performance of a considerable number of experiments. In addition, rate effects cannot be adequately accounted for. Consequently, in terms of usefulness over a wide class of materials and circumstance, Miner's theory is preferable.

Several researchers (31, 32, 33) have related the rate of crack growth to the localized energy and elastic stress conditions existing at the crack tip. The expression obtained when this fracture mechanics approach is used can be given in general form as

$$\frac{dc}{dN} = Ac^k\sigma^l \tag{2}$$

where

A and k = constants,
c = crack length,
σ = stress at tip of crack, and
N = number of load cycles.

The constants k, σ, and l are dependent on the properties of the material tested and on the boundary conditions of the problem in question (31, 32, 33). Paris and Erdogan (32) found that the use of values 2.0 and 4.0 for k and l respectively yielded good agreement with experimental results. Paris (78), by considering the energy dissipated per cycle of load application, dw/dN, as being proportional to dc/dN, attempted to relate the rate of crack growth to the stress intensity factor, K.

It is evident that these analyses have attempted to take rate effects into account in an indirect manner. When conditions of fracture are brittle in nature, then these analyses are accurate. However, in the presence of tearing action, the property of the material changes with time and thereby affects the corresponding response behavior to application of load, and such analyses cannot account for this kind of behavior. Despite these shortcomings, analyses of this type are attractive in the sense that the fatigue process has been linked to microphenomena on a phenomenological basis.

In order to take rate effects, order effects, and prior history effects into account, Dong (35) postulated a cumulative damage theory to predict the life of a material under any arbitrary loading history. Under isothermal conditions, the mathematical expression obtained is

$$l(t) = f\left[\gamma_{ij}(\tau)\right]_{\tau=-\infty}^{\tau=t} \tag{3}$$

where

l(t) = life remaining in the material at time t after damage has accumulated during $-\infty \le \tau \le t$,
τ = generic time,
t = present time,

γ_{ij} = any set of variables that can be used to describe loading history, and
f[ ] = damage functional.

This theory can account for the effect of prior history and sequence of events in damage behavior because the damage functional can be represented by an infinite series expansion of hereditary integrals of the linear and nonlinear types.

The damage behavior of any rate-dependent or rate-independent material can be predicted under any arbitrary loading history by using this approach. In the fatigue loading mode the Miner and Williams theories become special cases of Dong's concepts. This concept shows that the inability of the Miner and Williams theories to demonstrate the influence of prior history and sequence of events on failure is due to the restrictive form of their damage kernels.

The discussion on the damage created within a material in the repeated loading mode suggests that the manner of damage accumulation is a consequence of the inhomogeneity of the engineering materials and structures. Under load, various regions of stress concentration exist within the material; and because, of its inhomogeneous nature, a distribution of strengths is created such that some regions are weaker than others. When the strength of a weak region is exceeded it is quite possible that a crack may initiate and cause a redistribution of stresses with attendant crack formation in other regions. As the load is repeatedly applied, they propagate and grow to a size that eventually renders the material or structure unserviceable. When this event occurs, fatigue damage is completed.

Summary of Failure

In light of the several observations that have been made in regard to cumulative damage in the repeated loading mode, there are a few substantial points to consider in the course of developing a cumulative damage theory for highway pavements.

1. Damage is a function of the inherent inhomogeneity of materials and structures; its initiation, progression, and attainment of a critical magnitude are, therefore, stochastic processes.
2. For a given temperature, damage sites are nucleated under unique stress or strain conditions within the material. They propagate under stress or strain states different from initial conditions until a critical state is reached.
3. The state of damage at any time is a function of the material property and load history—i.e., damage is not unique; it is a function of stress level and microstructural changes within the material.
4. Assumptions have to be made regarding the manner in which damage propagates and regarding the parameters used to delineate its progression. The damage surface is essentially exponential in most materials, but the characteristics of the surface have to be determined from the stress and microstructural conditions existing in the material under load. For instance, if stress, strain, time, and temperature conditions within the material are such that brittle fracture is warranted, then the rate of damage accumulation is one of fast growth to failure. If a tearing kind of fracture is warranted, gradual accumulation of damage is experienced.

The next question that arises in view of the basic premise of this investigation is, Is it possible to develop a cumulative theory of damage for pavement structures comprising different engineering materials and utilizing the basic concepts of damage progression presented earlier? Such a theory may be able to bridge the rather wide gap between states of loading in the field and the relatively simple experiments on a mathematical model of the structure. The ability of the structure to adjust itself to these loadings should yield in symbolic terms, with some degree of reliability, the relationships between the external loadings and the physical constants that measure the competence of the system. To do this, we must know how a pavement fails in practice. The information, thus collected, must be interpreted in the light of the failure mechanisms governing the performance of the materials composing the pavement. When this is done, an adequate failure theory will begin to emerge. To this end the performance of a pavement structure in a repeated loading environment is examined in the next section within the context of internal damage development.

An engineering system in which damage or the failure of a component results in a decrease in the level of performance rather than the abrupt incidence of total failure may be called a "structure-sensitive system." For these systems, internal damage develops within an operational environment, over a given time period, and the failure is the ultimate condition that results from a loss in performance. Thus, failure is the extent of the damage that has been accumulated as a consequence of structural deterioration over a range of stress, strain, time, and temperature conditions in an operational environment.

The performance level of a structure-sensitive engineering system in an operational environment may be defined as the degree to which the stated functions of the system are executed within the environment. This level at any point in time is dependent on the history of the magnitude and the distribution of the applied load, the quality of the construction and materials used, their spatial distribution, and the extent to which proper maintenance practices are executed during the entire life of the facility. The reliability of the system consequently depends on these parameters, which are, in turn, dependent on the variabilities of nature. In other words, the performance level of the system at any instant has a probability of occurrence and a frequency distribution of values associated with it.

Figure 1 shows that the performance of the system diminishes in some way until an unacceptable level is attained. This behavior occurs as a result of the combined action of the load and the weather in a given environment. In this environment, pockets of local distress develop within the system (e.g., stress-induced inhomogeneity and corrosion), propagate in a manner that depends on the composition and spatial distribution of the structural materials, and bring about a loss in structural integrity with time.

The base level VP shown in Figure 1 represents an unacceptable level of performance, as determined by the users and engineers of the facility. The time within which the performance drops to this level characterizes the time at which the extent of damage in the structure becomes intolerable. At any instant of time t_i, P_i represents a level of performance, and associated with it is the degree of damage D_i developed within the period $t = 0$ to $t = t_i$. At time zero, the performance of the system is presumably 100 percent of its initial value, inasmuch as it has not yet been put into use. At the time when the internal damage becomes intolerable, the performance of the structure is zero percent of its initial value. The integrity level of the system at any time is, therefore, one minus the amount of damage accumulated within that time

$$P_i(t_i) = 1 - D(t_i) \tag{4}$$

In view of the previous discussion on damage it is obvious that the quantities P_i and D_i are probabilistic in nature. Thus, depending on the temporal and spatial distributions of damage within the structure the real instantaneous performance level will be on, below, or above the drawn curve. This means that each point on the curve has a probability of occurrence and a frequency distribution of values associated with it, and this fact must always be acknowledged.

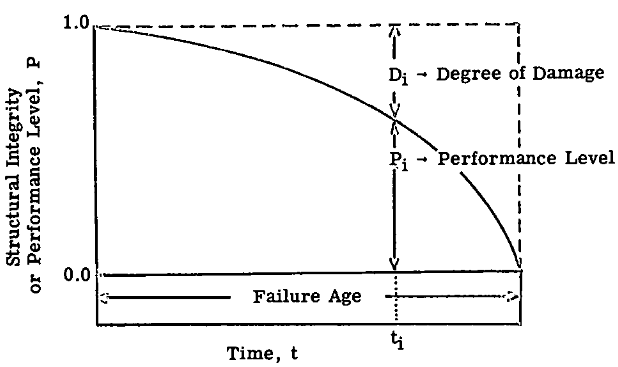

Figure 1. Two-dimensional simulation of the performance of a structure-sensitive engineering system.

The preceding discussion was conducted in the two-dimensional domain; the real picture is, however, more complex. The observed response of the structure depends on rate effects (36, 37), the position and magnitude of the applied load (38), climate (39), materials type, previous traffic history, temperature (38), and constructional variables (41). It can, therefore, be linear or nonlinear depending on the manner in which these variables combine. If the system behavior can be characterized as linearly or nonlinearly elastic, plastic, or viscoelastic, the response will have similar characteristics. Then, at any point t_1 in time a "performance surface" that is a function of these variables exists such that its inverse is a "damage surface."

Within this context, at any instant and at some point on the surface, a prediction of the percentage of total life already used and that remaining within the structure can be made. Therefore, the damage surface as well as the performance surface is n-dimensional parameters playing a significant role in its determination. The development of such a surface is not immediately possible. This, however, does not mean that the problem is intractable, because the possibility of reducing n may exist. In fact, such a technique is used in the development of yield surfaces for metals (42).

All the significant factors that have a role to play in internal damage progression can be generally accounted for, providing that they are translated, through the properties of the layer materials and the response behavior of the pavement structure for a given quality of construction and maintenance operations, into stress and strain quantities. In other words, the magnitude and type of stress and strain concentration (tensile or shear) within the pavement structure are a function not only of the characteristics of the applied load but also of the spatial distribution of layer material properties and local defects. A knowledge of the material properties yields information on the kind of structural response to expect. From such information postulations can be made about the manner of internal damage progression. This technique considers the two most significant structural properties, material properties and response behavior, which reflect the influence of all the others. This technique can, therefore, be used to classify pavements into three broad groups (frictional, flexural, and frictional-flexural) so that the stress-strain parameters of damage progression in each group can be identified.

The frictional type of pavement is composed of granular materials in which load transfer occurs at interparticle contact points by purely frictional action. The deformation that takes place under load is purely of the shear or flow type, and, for each application of the load, a permanent deformation results. Such pavement structures generally require a thin type of wearing course that can deflect conveniently with the rest of the structure under repeated loading. To protect the underlying materials, the wearing course should possess good ductile properties as opposed to brittle properties because toughness in this case is more important than tensile strength. However, when the deformation becomes excessive, cracks may appear in the surface because of the randomly distributed cumulative shear action in the subgrade. Therefore, in a frictional type of pavement, damage can be considered to develop as a result of shear action. Consequently, the damage parameter must somehow be associated with shear stresses and shear strains.

In a flexural type of pavement, the materials in the layers are capable of resisting the applied load through the action of tensile stresses that develop as a result of the flexing action. This implies that bending is the only mode of deformation and, upon the repeated application of load, repeated flexing results. In such a pavement, fatigue action is important; and, though the overall shear support of the components is adequate, cracks develop very early because of the accumulation of tensile strains. These propagate slowly or rapidly in a random manner depending on the porperties of the layer materials and the rate of the repeated flexing action. The fatigue properties of the materials in the layers are, therefore, a prime concern during the design state of such facilities. Damage in such pavements is propagated in the fatigue loading mode under the action of tensile stresses and tensile strains.

The third type of pavement possesses frictional and flexural materials. Its structural integrity under repeated load is impaired by the destructive tensile and shear action manifested within the layer components. It is conceivable that, if one action, tensile or shear, should dominate in creating damage within the structure, failure would occur in that mode. On the other hand, it is also possible that both actions may play a significant role during the life of the facility depending on environmental conditions. The damage parameter is, therefore, associated with both tensile and shear stresses and strains.

This classification makes possible the tractability of the damage progression within pavement structures, and one can generally say that the damage buildup occurs in three different modes, described in the following. When the behavior of the pavement structure is completely frictional, damage initiates and progresses by plastic or shear flow until the appearance of surface cracks terminates or aggravates the situation. When flexural behavior is pertinent damage, initiation, and progression occur by the development and growth of internal cracks, under the action of tensile stresses and strains. However, in the frictional-flexural type of pavement, the damage initiates and progresses by shear flow or by crack growth or by both. Consequently, a pavement structure may show signs of distress either from the independent action of excessive deformations, from the isolated action of fatigue, or from both failure mechanisms working together. This indicates that, in order to analyze the response behavior of a pavement structure and to predict the failure behavior, we must develop a number of models that would account for such behavior in a given traffic and climatic environment.

At the present time, the following three models seem to be appropriate:

1. A model is needed for the representation of the linear and nonlinear behavior of paving materials;
2. The pavement system must be modeled in terms of the geometrics of the applied load and the structure so that use of the former model within such a framework will aid in the prediction of the developed stresses and strains in a given environment; and
3. A model that must be capable of handling linear and nonlinear damage behavior should be developed.

Finally, to achieve realistic predictions requires that these models be combined in a probabilistic manner, inasmuch as the progression of damage as has been demonstrated is stochastic in nature.

Because the objectives of this study are to provide a better understanding of mechanisms of damage and distress in pavement structures, the remainder of the discussion is devoted to item 3 and the stochastic nature of the problem.

Models for Pavement Damage

The pavements discussed in this section belong to the frictional-flexural group and are therefore representative of many current pavement sections. In this type of structure fatigue damage may occur in the surface layer when it behaves in a flexural manner.

The occurrence of fatigue in pavements has been observed or noted for a considerably long period of time. Porter (55) in 1942 observed that pavements do, in fact, undergo fatigue. In 1953, Nijboer and Van der Poel (56) related fatigue cracks to the bending stresses caused by moving wheel loads. Hveem (57) also correlated the performance of flexible pavements with deflections under various repeated axle loads. The AASHO and WASHO tests (39) confirmed these observations by relating the cracking and initial failure of pavements to repeated loading of the type discussed by Seed et al. (58).

The field observations of this kind of behavior led to laboratory investigation. Many researchers have conducted laboratory experiments to determine the fatigue properties of paving materials and to investigate the possibility of extrapolating laboratory results to existing field conditions. To this end, Hennes and Chen (59) conducted tests on asphalt beams resting on steel springs and subjected to sinusoidal deformation with a variety of constant amplitude magnitudes. They discovered that, as the frequency of application is increased, the creep-rupture compliance of the material decreases. Similar tests conducted by Hveem (57) on beams cut from actual pavements yielded the same results.

Monismith (60) in his tests on asphalt beams supported on flexible diaphragms mounted on springs under constant stress amplitudes discovered that increases in the stiffness of the material resulted in corresponding increases in fatigue life. Sall and Pell (61) conducted similar tests from which the tensile strain to failure, ϵ_T, versus the number of cycles to failure or fatigue life, N_f relation, was found to be $N_f = 1.44 \times 10^{-16} (1/\epsilon_T)^6$. They further found that this expression does not vary with temperature, rate of loading, and type of asphalt. These results are not surprising, because one should expect such factors to affect the developed stresses and not the strains through the stiffness of the material. For the mixtures tested, no endurance limit was observed up to 10^8 application, as is to be expected, because the mode of failure is one of crack initiation and propagation to failure at each stress level. The general conclusion arrived at by several authors from such tests indicates that the fatigue life of an asphaltic paving material is a function of several variables: tensile strain level to which the specimen is subjected, amount of asphalt, and age, temperature, stiffness, density, and void ratio of the mixture.

Another important factor in such tests is the mode of loading. In controlled stress tests, for example, fatigue life increases not only as the stiffness of the sample increases but also as the temperature decreases. However, in strain-controlled tests, the fatigue life decreases as stiffness increases. For this test, at low temperatures no change is observed in fatigue life, and as temperature increases the fatigue life increases as well (63, 64, 65). Controlled stress and strain behavior can be explained from a consideration of either the time-temperature superposition principle or the amount of energy stored in the sample when such tests are performed. In controlled-stress tests the minimum energy stored per load repetition can be achieved by minimizing deflection and causing a resultant increase in fatigue life. In controlled strain tests, the reverse is true. This implies that, for a specimen of a given initial stiffness and initial strain, failure under a controlled stress mode of loading will occur sooner. Therefore, when extrapolating laboratory results to field conditions such considerations play a significant role. Monismith (67) through the use of a mode factor suggested that, for surface layers less than 2 in. thick, the controlled strain mode of loading results; whereas for those layers 6 in. thick or greater, the controlled stress mode of loading is applicable. For thickness between these, an intermediate mode of loading is appropriate.

Tests have also been performed on granular and other paving materials to determine the significant characteristics of their behavior under repeated loading (58, 66). The approach is empirical; however, it points up the important fact that the response of granular and treated materials in pavement sections depends on the characteristics of the applied loading, the material, and the existing confining stress.

In an attempt to apply the experimental results to predict the occurrence of damage in a real pavement, Deacon and Monismith (69) suggested a modification of the Miner theory of linear summation of cycle ratios. They pointed out that such an approach has the desirable features of procedural simplicity and a wide range of applicability to different types of compound loading. Their analysis, however, is rather difficult to interpret; also, the sequence of events and prior history cannot be accounted for in such an approach.

Majidzadeh et al. (82) in their recent work have shown that crack initiation and propagation in asphaltic mixtures can be predicted by using the fracture mechanics approach. They have found that the critical stress intensity factor, which is a function of the material's elastic properties and the driving force at the tip of the crack, is an inherent property of the asphaltic mixtures at low temperatures. Because the crack geometry affects the value of the stress intensity factor, the authors suggest an analytical technique for its calculation for some practical cases.

Stochastic Nature of Damage

Factors contributing to the initiation, propagation, and accumulation of pavement damage can be divided into three categories: (a) material properties and pavement geometry, (b) loading variables, and (c) climatic conditions. A substantial variability is associated with the measurement or specification of each of these factors, and as a result the pavement response is stochastic. To account for these variabilities requires that the damage model be able to yield statistical estimates of the pavement response. In other words, the model should be able to estimate the probability that damage will occur in a certain mode. To achieve this, we can use simulation techniques. The following discussion describes the possibility of using one such technique, the Monte Carlo method, to predict the stochastic behavior of damage accumulation.

Materials and Environmental Variabilities

The behavior of a material in a given environment can be represented by a set of responses, R_i. The material itself is defined by a set of relevant properties, Y_k, and the environment can be specified by a set of conditions, X_j.

In the deterministic approach, it is usually assumed that a functional relationship exists between each response term and the associated material properties and environmental condition. Material properties will also vary systematically with environment. So,

$$R_i = g_i(Y_1, \ldots, Y_k, \ldots, | X_1, \ldots, X_{ji}, \ldots, X_n) \tag{5}$$

$$Y_k = \nu_k(X_1, \ldots, X_j, \ldots, X_n) \tag{6}$$

However, both material properties and environmental conditions are subject to considerable variability in a random manner over fairly wide ranges. When environmental conditions are correlated, i.e., when there is an interaction between these parameters such as the interaction of moisture and temperature and the effect of one on the other, their joint frequency distribution $f(X_1, X_2, \ldots, X_n)$ will yield the density function as the necessary data to be input for the environmental conditions. If they are not correlated, their independent frequency distributions could sufficiently define the environment. The vectors X_j are, therefore, treated as random variables with probability density functions, f_{x_j}, and associated cumulative distributions, F_{x_j}.

Material properties are inherently variable, and the terms, Y_k, are also considered to be random variables with density functions, f_{Y_k}, and cumulative distributions, F_{Y_k}.

Because the material properties are dependent on the environmental conditions, statistical correlation is implied by Eq. 6. Complete information of inherently correlated material properties can be given by the joint density function $fY_1, Y_2, \ldots, Y_L$ rather than by the density functions f_{Y_k}.

Variability in material properties and environmental conditions implies variability in the material behavior or response. Variability in material behavior is represented by a set of density functions, f_{R_i}, or alternatively by the cumulative distribution, F_{R_i}.

To evaluate f_{R_i}, one should have data available about the density functions f_{x_j} and f_{Y_k}. Even if these density functions are somehow evaluated, considerable difficulty can arise in determining f_{R_i} by analytical methods. Such difficulties can be encountered if f_{x_j} and f_{Y_k} are not normal and the equations giving the functional relationships of the three basic vectors are not linear. In these cases a numerical solution can be obtained by the Monte Carlo method.

Monte Carlo Simulation

The Monte Carlo technique is a simulation method for the evaluation of the cumulative distribution, F_{R_i}, in an algorithmic form suitable for computer programming (83). The method is probabilistic in its approach and considers a conditional probability distribution of the form

$$F_{Y_k}|x_j \; (Y_k \leq y_k | X_j = x_{ji}), \; j = 1, 2, \ldots, p \tag{7}$$

where x_{ji} is any set of values of X_j from populations with cumulative distributions F_{x_j}. Similarly, the inherent interdependence of material properties can be taken into account in the same algorithm; i.e.,

$$F_{Y_k}|x_j \; (Y_k | X_j) \times F_{x_j} \; (X_j) = F_{Y_k} \; (Y_k) \tag{8}$$

The algorithm involves an iteration process from which n number of samples is drawn for the values of R_i. From the sample of n, simulations, histograms, means, variances, and percentage points can be obtained. If the number n of the simulation is very large, the histogram can accurately represent the continuous distribution of the parent population.

This simulation method is a simple numerical method that gives statistical answers to specific problems that are not amenable to analytical procedures because of their inherent complexity and interacting factors. This method is approximate in nature, but a high degree of accuracy can be achieved if the number of simulations is sufficiently large. The sufficiency conditions here depend on the statistical data available or required.

In this case the decision as to how many samples to be drawn out should be preceded by something like sensitivity studies. Several techniques have been developed based on such studies. These are basically variance reducing techniques to increase information in the "interesting regions" of the distribution F_{R_1} and, hence, to decrease the information in the noninteresting regions or ranges.

The other factors that have an influence on the cumulative frequency distribution are the probability density functions, interactions, and correlations of the parameters involved, i.e., the environmental variables and the material properties. In the case of interacting parameters, a joint density function can be used instead of the single density functions.

The probability density functions of the different parameters that are being simulated can generally be obtained by some statistical tests. Sampling from the real, statistically measured distribution is preferable to obtaining samples from the assumed distribution because the more realistic these density functions are, the better the results of simulation are. However, if the statistical data are lacking for density functions of the parameters under consideration, special care should be given to making assumptions for such density functions. This could be done by looking into the literature for statistical representation of the same or similar parameter.

Summary of Pavement Damage

The results of the review of literature on pavement damage due to the mechanical loading indicate that, in order to develop a general damage function for pavements, the pavement system may be considered as a black box that possesses some unknown properties and is subjected to some variable inputs (load and environment). The properties of the black box are dependent not only on the distribution of the inputs but also on the materials and geometrical configuration of the structure. The outputs are the responses of interest such as the deflection, the extent of cracking, and so forth.

In such structures, damage accumulates, and either a modified form of Dong's general theory or a modified form of Miner's linear law is needed to account for accumulation of damage. Any damage concept developed for highway pavement should be adaptable to account for the stochastic nature of the problem. A direct probabilistic method should be used if possible; otherwise, a method using simulation techniques should be developed.

PAVEMENT CUMULATIVE DAMAGE MODEL

Formulation of Damage Law

To develop a general damage law one must relate the input variables (loads, temperature and time) to output variables (deflections and cracks). The relationship sought is generally difficult to obtain, and its application to each specific case requires a great many modifications. The complexity of the required general relation can be simplified by handling the damage formulation in two steps. The first step yields the stress and strain fields within the system. Stress, strains, and displacements are called the primary responses and are then used as inputs to the second step in the model, which yields damage parameter or limiting responses. This separation of the damage formulation into two independent parts assumes that the limiting responses are dependent solely on the primary responses, a fact that seems to be supported in the literature.

Primary Response—The geometrical model that is used in this study is a semi-infinite half space consisting of three distinct layers. It is assumed that each layer has distinct material properties that can be characterized as linear elastic or linear viscoelastic. The load is considered to be uniform, normal to the surface, and acting over a circular area.

An assumption of incompressibility is made so that one constitutive relationship is sufficient to define the viscoelastic equation of state for each layer. This constitutive equation is assumed in terms of a viscoelastic equivalent to the elastic compliance. That is, for the ith layer,

$$\frac{1}{E}:\ (\text{equiv.}) = \left[D_i(o)\,(\ ) - \int_o^t (\ )\, \frac{\partial D_i(t-\tau)}{\partial \tau}\, d\tau \right] \tag{9}$$

where $D_i(t)$ = the creep compliance of the ith layer.

To obtain the viscoelastic solution for the stresses and displacements for other loading conditions, we use the correspondence principle (72). Using this method, Elliott and Moavehzadeh (71) have analyzed three loading conditions: a stationary load, a repeated load, and a load moving at a constant velocity. In each of these three cases, the environmental conditions were assumed to remain constant. In this study the results of their study are used and extended to account for varying environmental conditions.

Limiting Response—The structural damage of a pavement structure is divided into two parts that are not necessarily independent of each other: excessive deformation and cracking. The excessive deformation can be directly predicted by any primary model of a multilayer system that has accumulative capabilities (71), provided that it is modified to account for the stochastic nature of the environmental factors. Cracking is assumed to arise mainly from fatigue behavior, and its accumulation thus can be measured in different manners. For example, one can relate the crack nucleation and growth to specific combinations of the primary responses such as the stress intensity factor K (82) and then evaluate the probabilities of having a given distribution of cracks. Another method is to use a more phenomenological approach and represent the accumulation of cracking by a damage functional F(t), which depends on the past histories of the stress and strain tensors. With some assumptions of continuity this functional can be expanded into a series of multiple integrals (35):

$$F(t) = \int_o^t K_1(t, s)V(s)ds + \int\int_o^t K_2(t, s_1, s_2)V(s_1)V(s_2)ds_1ds_2$$

$$+ \ldots + \int \ldots \int_o^t K_n(t, s_1, \ldots, s_n)V(s_1) \ldots V(s_n)ds_1, \ldots, ds_n \tag{10}$$

The measure of damage F(t) is not uniquely defined. The damage may be measured by the density of cracking or by the value of dynamic modulus of the layer materials at a given frequency because this modulus decreases as the density of cracking increases (87). The creep compliance of the material can be used as a measure of crack propagation (82, 92), or the number of remaining cycles before complete failure under a given mode of loading can be used for this purpose (85). Any of these measures can be used, and it is convenient to normalize them so that the damage functional equals zero when the material is intact and increases to one at failure. In Eq. 10 V(s) is a function involving stress or strain invariants or both, and s is an arbitrary parameter that may have a meaning of time or cycles. This representation of the damage functional is general and accounts for accumulation of damage as well as recovery processes such as healing and accumulation of aging effects.

The review of literature showed that for various asphaltic and bituminous mixtures failure envelopes were related to a strain measure. In the general case of triaxial loading conditions the strain measure should be expressed as a combination of invariants. In the absence of results of triaxial tests, we will use the derivative of the major principal strain as a strain measure in the damage functional. Thus,

$$F(t) = \int K_1(t, s)\dot{\epsilon}(s)ds + \int_o\int^t K_2(t, s_1, s_2)\dot{\epsilon}(s_1)\dot{\epsilon}(s_2)ds_1ds_2 + \ldots \tag{11}$$

where ( ) represents a differentiation with respect to the argument. When s has a meaning of a time, this expansion is similar to the representation of the time response of a nonlinear viscoelastic material; whereas when s has a meaning of a cycle, it may be related to the dynamic representation of a nonlinear viscoelastic material and may be determined as a transfer function of a system subjected to a cyclic loading. Dong (35) has shown that, in the latter case, making these integrals discrete results in Miner's law (27).

This expansion is simplified by making an assumption that three different damage processes may be recognized: a damage process depending on the number and amplitude of cycles, a healing process (or a recovery process) depending on the elapsed time since the damage was created, and an aging process wherein the materials properties are changing with time. The damage functional may now be written as

$$F(t) = \int_0^t K_1(s, t-s)\epsilon(s)ds$$

$$+ \int_0 \int^t K_2(s_1, t-s_1, s_2, t-s_2)\epsilon(s_1)\epsilon(s_2)ds_1 ds_2 + \ldots \qquad (12)$$

This equation implies that the kernels are functions of the running time s (cumulative and aging processes) and of the lapse of time t - s (recovery process).

In a first approach to the problem, the second and higher order kernels will be neglected in the damage expression. We will further assume that the first order kernel sought may be factorized as

$$K_1(s, t-s) = K_{CUM}(s)\, K_{REC}(t-s, s) \qquad (13)$$

i.e., the cumulative and recovery processes are independent. The aging process is included in both K_{CUM} and K_{REC} through the dependency on the time s.

In order to determine these kernels we must choose a measure for the damage and normalize it as mentioned. Let N be the number of cycles to failure (i.e., inadmissible density of cracking) under a given type of random load during a relatively short time (no aging or recovery takes place). A damaged material will undergo only N′ cycles under the same conditions before failing. The amount of damage is represented by (N - N′)/N. N and N′ can be measured on control specimens. Note that in this case F(t) is not a measure of the amount of cracking but is a function of it.

Cumulative Kernel—For a small period of time during which there is neither aging nor damage recovery we have for the increment of damage, $\Delta F(\tau)$

$$\Delta F(\tau) = \int_0^S K_{CUM}(S-s)\frac{\partial \epsilon}{\partial s}\, ds \qquad (14)$$

Dong (35) proved that, if s is the number of cycles of a given strain amplitude, Eq. 14 is identical to Miner's representation; i.e.,

$$\Delta F(\tau) = \sum_{i=1}^{m} \frac{dn_i(\tau)}{N_i(\tau)} \qquad (15)$$

where $dn_i(\tau)$ is the number of cycles of amplitude $\Delta\epsilon_i$ that are applied at time τ, and $N_i(\tau)$ is the number of cycles $\Delta\epsilon_i$ that would cause failure. Hence, the general expression for the damage becomes

$$F(t) = \int_0^t \Delta F(\tau)\, K_{REC}(t-\tau, \tau)\cdot d\tau$$

$$F(t) = \int_{0}^{t} K_{REC}(t - \tau, \tau)\left(\sum_{i=1}^{m} \frac{dn_i(\tau)}{N_i(\tau)}\, d\tau\right) \tag{16}$$

The aging process is accounted for in the dependency of K_{REC} and N on the time τ.

For a uniaxial case we will consider a random stress history to be composed of a mean value and a cyclic component. In the triaxial case these may be replaced by a mean value of the hydrostatic stress and a cyclic component of the octahedral stress. If sinusoidal stresses with constant amplitude are applied to various specimens of a material, a fatigue envelope (S-N curve) is usually obtained in the form of a stress or strain amplitude versus the number of cycles to failure. Miner's law may be applied to such diagrams to predict the results under varying amplitudes. Because this envelope is found to be generally independent of temperature and rate of loading when it is given in the form of strain amplitude versus number of cycles to failure, we will concentrate on the use of such diagrams. These diagrams and Miner's law, however, do not account readily for the order in which successive loads are applied and the effects of varying amplitudes. The order in which the loads are applied will be accounted for implicitly because strain amplitudes are obtained as the primary responses of the three-layer viscoelastic model, and thus they are functions of the sequence of loading. To take into consideration the effects due to varying amplitudes and nonzero mean, we can use an approach suggested by Freudenthal and Heller (84) and based on the statistical character of fatigue. The basis of this statistical evaluation of the results is the three-parameter distribution function of the smallest values (84). This function derives from the assumption of a distribution function of the strength (or strain at failure) of cohesive bonds in the material and a distribution function of the induced stresses (or strains) when a macroscopic stress (or strain), S, is applied to the structure. The probability function, P(S), of the strength (or strain at failure) of the material is obtained as a convolution of the two previous distribution functions. The relation between the mode of loading P(S) and N determines the trend of a theoretical S-N diagram. Based on the determination of a single S-N diagram obtained for a given mode of loading (e.g., sinusoidal with constant amplitudes), we can predict the probable S-N diagrams for other modes of loadings. Freudenthal and Heller (84) related the effect of the change in the load spectrum to the changes in the S-N diagram.

This results in a modification of Miner's law to include an interaction factor $\omega_i > 1$ to account for interaction effects of various stress amplitudes. Thus, Miner's law becomes

$$\sum_{i=1}^{m} n_i/\omega_i\, N_i = 1 \tag{17}$$

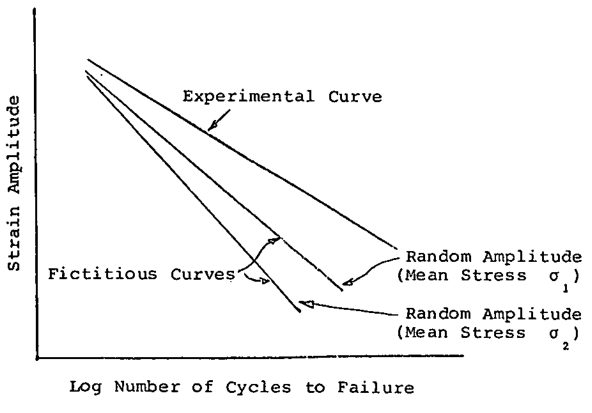

Figure 2. Failure envelopes.

where ω_i depends on the load spectrum and results in the fictitious envelope shown in Figure 2. The cumulative process can therefore be given by an expression such as

$$\sum_{i=1}^{m} \frac{dn_i}{\omega_i N_i\,[\Delta\epsilon(\tau),\, \tau]} \tag{18}$$

where τ indicates that the number of cycles to failure may vary because of aging and that the envelope is to be determined for different values of τ. The increment of damage is also a function of the average strain amplitude applied during the increment of time τ.

The determination of $N_f(\Delta\epsilon)$ can also be derived from the knowledge of the mechanisms of cracking. For example, it can be determined through the use of the concept of stress intensity factors (82).

Recovery Kernel—The recovery kernel, $K_{REC}(t-\tau, t)$, is a function of the time, $(t-\tau)$, elapsed since the application of the damage increment and of the age of the material. From Bazin and Saunier's paper (85) it is apparent that healing requires the presence of a minimum compressive stress. Thus, we will assume that the argument $t-\tau$ can be replaced by $t^* - \tau$ where

$$t^* = \int_{\tau}^{t} H[\sigma(s) - \sigma_{min}]ds \qquad (19)$$

H[ ] is the Heaviside step function, which is equal to one when its argument is positive and equal to zero elsewhere. σ_{min} is the minimum compressive stress that triggers healing. Thus, $t^* - \tau$ is the accumulated time during which a minimum compressive stress is present.

To determine $K_{REC}(T)$ we should give two identical specimens (or sets of specimens) the same amount of damage, F. F is determined by testing one of the two specimens (control specimen) and measuring the amount of damage that should still be applied to fail the specimen. The second specimen is left to rest for a time, T, and then failed to determine the amount of recovery, $K_{REC}(T)$.

Aging—Aging is accounted for through changes in the characteristics of the constitutive equation as well as in the cumulative and recovery kernels.

Input Requirements

Materials Characterization—For the determination of the damage function, F, it is important to determine the stress or strain invariants or both or, as in Eq. 19, to determine the major principal strains. To do this, we must determine the properties of the materials in the layers. These properties are generally dependent on the manner in which they are prepared and constructed, their thickness and confining stress, the rate of loading, and the history of the environmental variables. Because all of these factors are statistical quantities, we must expect the properties to also be statistically distributed within the layers.

The materials properties assumed to be pertinent here are the compliances or creep functions, Poisson's ratio, and the height of the layered materials. Poisson's ratio and the height of each layer are considered as deterministic quantities. Although the height of the layers can change with different structural sections, Poisson's ratio is set equal to one half for all sections. The properties of the material for each layer will be represented by a creep compliance function for a viscoelastic layer and by a creep compliance for an elastic layer. For a viscoelastic layer the following representation will be assumed

$$D_j(t) = D_{E_j} + \sum_{i=1}^{n} G_i^j e^{-t\delta_i} \quad j = 1, 2, 3 \qquad (20)$$

where

j = layer number;

$D_j(t)$ = value of creep function at time t;

D_{E_j} = zero time value of the creep function, i.e., $\sum_{i=1}^{n} G_i^j = 0$;

G_i^j = coefficient in Dirichlet series $\sum_{i=1}^{n} G_i^j e^{-t\delta_i}$;

δ_i = exponent in exponential term corresponding to coefficient G_i^j; and

$\delta_n = 0, \ldots, D(\infty) = D_{E_j} + G_n^j$.

To include the statistical characteristics of the properties in the analysis requires that a method similar to that described by Soussou and Moavenzadeh (86) be used. In this method, a random loading is used as an input in the tests designed to determine the creep compliances. The resulting functions are least square approximations of the social properties.

History of the Environment—The main measurable quantities reflecting the influence of the environment are temperature and humidity. The temperature within the system will be assumed to be uniformly distributed. Later stages of the study may introduce the spatial distribution of temperature as a function of the atmospheric temperatures and wind condition. Data are available on such distributions, and analytical means of computation exist (88), but the present models for viscoelastic layered systems do not have this capability. The same observations as made for temperature can be made for the moisture or humidity distribution within the system. The history of environment will be generated in a random manner so as to approximate the climate in a given area.

Many viscoelastic materials were shown to be "thermorheologically simple," i.e., to fulfill the time-temperature superposition principle. Similar principles of superposition were found to be true for other types of environmental changes (89). Thus, if $\phi(t)$ is the value of the environment at time t and ϕ_0 is the reference value of the environment, viscoelastic properties at $\phi(t)$ may be derived from viscoelastic properties at ϕ_0 through different operations of scaling. A general expression is

$$D[t, \phi(t)] = \alpha[\phi(t)] + \beta[\phi(t)]\, D\{\gamma[\phi(t)]\, t, \phi_0\}$$

where $\phi(t)$ is a function of the values of the temperature or moisture content, α is a vertical shifting of the creep compliance, β is a vertical scaling of the transient response, and γ corresponds to a horizontal shifting for a semilog plot (change in time scale). These techniques of scaling apply well to a wide variety of viscoelastic materials, and they allow for an easy introduction of the environment effects in viscoelastic analyses.

This representation is used to describe the effect of changes of properties with temperature and moisture. Moreover, because the repeated load program can be directly related to the results of the stationary load program, these scaling techniques will be used to represent the response of the stationary load program for different values of the environment variables.

History of Load Applications—The traffic load intensity on a pavement system is statistically distributed in time, space, and magnitude. In this investigation, a single wheel load applied over a circular area is considered to be appropriate on the assumption that the equivalent-single-wheel-load concept is valid. The rate of load application and the magnitude of the load can be varied. The loading function is assumed to be a Heaviside characterized by a load duration and a period that is the time between two consecutive load applications. At this stage of the study, the load magnitude and the duration of the load are assumed to be constant. The period of the load is constant during a time period (day, week, month, and so on) and is a function of the number of load applications during that period.

Application to Highway Pavements

Primary Response—Because a major part of the problem of determining the damage functionals is to predict stresses, strains, and deflections under a random load and environment histories, it was necessary to modify the repeated load program (71) to give it this capability. The response, $P_s(t)$, of a linear viscoelastic system to any varying load, $P(t)$, may be written as

$$P_s(t) = \int_{-\infty}^{t} SR(t - \tau) \frac{\partial}{\partial \tau} P(\tau) d\tau$$

where SR(t) is the response to a step load (stationary load program). If we want to introduce the effect of the history of the environment, $\phi(t)$ where $\phi(t)$ may be the history moisture, and other environmental variables, the response is given by (71)

$$P_s(t) = \int_{-\infty}^{t} SR\left[t-\tau, \underset{s=\tau}{\overset{t}{\phi}}(s)\right] \frac{\partial}{\partial\tau} P(t) d\tau$$

and

$$SR\left[t-\tau, \underset{s=\tau}{\overset{t}{\phi}}(s)\right] = \alpha[\phi(t)] + \beta[\phi(t)]$$
$$\times SR\left\{\int_{\tau}^{t} \gamma[\phi(\theta)]d\theta, \phi_o\right\} - \int_{\tau}^{t} SR\left\{\int_{\tau}^{t} \gamma[\phi(\theta)]d\theta, \phi_o\right\} d\beta[\phi(s)]$$

where α, β, and γ are scaling factors that were described earlier and that are functions of the environment $\phi(t)$; and ϕ_o is the reference value for the environment. The determination of α, β, and γ is made by curve-fitting techniques (86).

When SR(t) is given in form of a series of exponentials

$$SR(t) = \sum_{i=1}^{N} G_i e^{-t\delta_i}$$

for a step load under a constant value $\phi(t)$, the response becomes

$$SR[t, \phi] = \alpha[\phi] + \beta[\phi]\left(\sum_{i=1}^{N} G_i e^{-t\delta_i[\phi]}\right)$$

whereas in the more general case of a variable environment history it becomes

$$SR\left[t-\tau, \underset{s=\tau}{\overset{t}{\phi}}(s)\right] = \alpha[\phi(t)] + \beta[\phi(t)]$$
$$\times\left(\sum_{i=1}^{N} G_i e^{-\delta_i t^*}\right) - \int_{\tau}^{t}\left(\sum_{i=1}^{N} G_i e^{-\delta_i s^*}\right) d\beta[\phi(s)]$$

The notation x^* is defined as

$$x^* = \int_{\tau}^{x^*} \gamma[\phi(\theta)]d\theta$$

At the present time only the temperature is considered in the variable $\phi(t)$, and it is assumed that the temperature remains constant for a time period and changes as a step function at the end of every time period. Thus,

$$SR\left[t-\tau, \underset{s=\tau}{\overset{t}{\phi}}(s)\right] = \alpha[\phi(t)] + \beta[\phi(t)]\left(\sum_{i=1}^{N} G_i e^{-\delta_i t^*}\right)$$
$$- \sum_{m=1}^{j} \sum_{i=1}^{N} \left(G_i e^{-\delta_i s^*_m}\right) \times \left[\frac{T(t_m) - T(t_{m-1})}{T_o}\right]$$

This formulation can be used to compute the primary responses due to arbitrary histories of loads and environment variables. In the present work we have adopted the concept of equivalent single wheel loads where the magnitude of the applied load is maintained constant. In the more general case the magnitude of the applied load will also present a statistical distribution. In the present analysis, however, the magnitude of the load, as well as the duration of its application, is considered to be constant. The load is described by

$$P(\tau) = \text{sine}^2\omega\tau[H(0) - H(\text{duration}) + H(1 \times \text{period}) - H(1 \times \text{period} + \text{duration}) + H(2 \times \text{period}) - H(2 \times \text{period} + \text{duration}) + \ldots]$$

where H is the Heaviside step function. Hence,

$$\frac{\partial P(\tau)}{\partial \tau} = \begin{cases} \omega \sin \omega\tau \text{ while a load is applied} \\ 0 \text{ otherwise} \end{cases}$$

The unit step response is described by

$$SR\left[t-\tau, \overset{t}{\underset{s=\tau}{\phi}}(s)\right] = \alpha[\phi(t)] + \beta[\phi(t)]\left(\sum_{i=1}^{N} G_i e^{-\delta_i t^*}\right) - \sum_{m=1}^{j}\sum_{i=1}^{N}\left(G_i e^{-\delta_i s^*_m}\right) \times \left[\frac{T(t_m) - T(t_{m-1})}{T_o}\right]$$

Based on results obtained by Glucklich (92), we will use the following as typical values in the computer program:

$$\alpha[\phi(t)] = 0$$
$$\beta[\phi(t)] = T(t)/T_o$$
$$\gamma[\phi(t)] = 1/a_T(t) = 10\left\{10{,}000\left[\frac{1}{T(t)} - \frac{1}{T_o}\right]\right\}$$

where T(t) and T_o are respectively the present temperature and the reference temperature in degrees Kelvin, and $a_T(t)$ is the present value of the shift factor. In this case we can write

$$SR\left[t-\tau, \overset{t}{\underset{s=\tau}{\phi}}(s)\right] = (T_j/T_o)\left(\sum_{i=1}^{N} G_i e^{-\delta_i t_j^*}\right) - \sum_{m=1}^{j}\sum_{i=1}^{N}\left(G_i e^{-\delta_i s^*_m}\right) \times \left[\frac{T(t_m) - T(t_{m-1})}{T_o}\right]$$

where t_j^* and s_m^* reduce to

$$t_j^* = \int_0^{t_j} \gamma(\tau)d\tau$$

and

$$s_m^* = \int_0^{s_m} \gamma(\tau)d\tau$$

Limiting Responses—The limiting responses that will be considered for a pavement are rutting, slope variance, and cracking. The first two are directly predictable from the primary response, but the latter requires the development of a damage model. Although cracking may result from a single load application, it is more often created by repeated loading, i.e., fatigue. The amount of damage created by fatigue is represented by the function F(t) defined previously.

The evaluation of F(t) requires the knowledge of the kernels involved. The form of these kernels was suggested in the review of the literature. The cumulative kernel is given by the fatigue envelope relating strain amplitudes and number of cycles to failure. This fatigue envelope is defined by a relationship of the form $N = K\ (1/\Delta\epsilon)^n$ (88), where N is cycles to failure at a particular strain level, K is of the order of 10^{-6} to 10^{-10} for various asphalt concrete mixtures, and n varies between 2.8 and 5. $\Delta\epsilon$ will be defined as the average difference between two consecutive peaks and valleys in the strain function. The strain level is the average strain level for the whole period K, and n can be given in terms of mean strain level (as is done commonly for metals). The recovery kernel is more difficult to obtain because of the scarcity of data. It is possible to use some experimental results such as those of Kasianchuk's (88), who reports the rate of healing and recovery of some asphaltic mixes.

Still fewer data are available for evaluating the effects of aging. This process is important to include because it accounts for some of the non-load-associated failures.

Systems Simulation—Figure 3 shows the steps involved in modeling the pavement system and simulating load and environmental histories. This section will describe the different steps involved in the formulation of this model.

The first stage in the model consists of dividing the time into periods during which the environment variable (i.e., temperature) is assumed to be constant. These time periods may be days, weeks, or months, depending on the assumptions made for the analysis. The duration of the load and its magnitude are assumed to be constant. The number of load applications for each time period is generated randomly so that it has given statistical characteristics, e.g., uniform distribution over a given range. Similarly, the value of the temperature for each time period is also a random variable that has a specific frequency distribution such as a normal distribution over a given range of temperatures.

The characterization of the materials properties yields environmental scaling factors α_i, β_i, and γ_i for the ith layer. Then the responses of the three-layer viscoelastic model to static loads at different values of the temperature, T, are curve-fitted to obtain the unit step response of the system at a reference temperature $SR(t, T_{ref})$, as well as scaling factors $\alpha(T)$, $\beta(T)$, and $\gamma(T)$ for the overall system.

In the present analysis, because there were no particular assumptions on the values of these coefficients for each layer, the values of $\alpha(T)$, $\beta(T)$, and $\gamma(T)$ were assumed to be those found for a particular sand-asphalt mixture (87). These values were assumed to be the same for both the deflection and the strain unit step response of the layered system. Note that the variability of the materials properties is not included, but this can be done by associating frequency distributions for each of α, β, and γ and the coefficients describing $SR(t, T_{ref})$.

The simulation program proceeds then to compute the total residual deflection at the end of the jth time period as well as the mean circumferential strain and average circumferential strain amplitudes during the jth time period. The strains are computed at the bottom of the top layer, and the strain amplitudes are computed as half the difference between successive peaks and valleys of the resulting strain.

These results are then readily related to the three principal measures of damage: rutting, slope variance, and cracking.

Rutting is measured by the amount of residual deflection. A large number of simulation runs yield a probability of occurrence of a maximum deflection before a given number of time periods. The variability of the materials properties can be easily accounted for in such results; however, for the present time deterministic values were chosen for the materials properties in order to keep the required number of simulation runs at a minimum.

Slope variance is measured by differential deflections. These differential deflections occur mainly because of variabilities in the system's properties. Therefore, it is

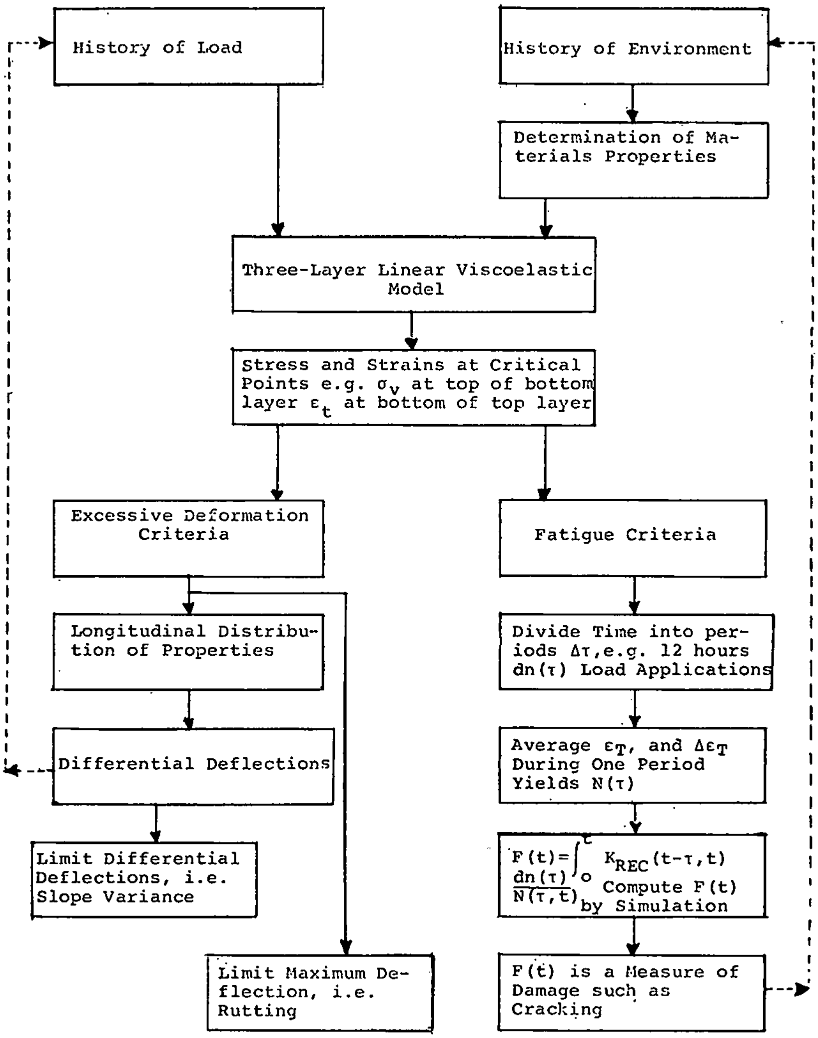

Figure 3. Modeling pavement system and simulating histories.

important to evaluate the correlations between the properties of the system at two points separated by a distance d (e.g., d = 2 ft). The knowledge of these correlation coefficients allows determination of the probabilities of having a given amount of differential settlement before the j th time period. This determination is done through a series of simulation runs representing each of the two points. Each of the points is assumed to be a three-layered, half-space system (i.e., the differential settlement is not accounted for) directly in the mathematical model.

Cracking is measured by the function F(t) as essentially a fatigue phenomenon. At the end of each time period, the increment of damage $\Delta F(\tau)$ is evaluated by using a Miner's type of law. The number of cycles applied during that time period is known, and $N(\tau)$ is obtained from the value of the average amplitude of the strain $\Delta\epsilon_t$ and its mean value ϵ_t. The total value of F(t) is obtained by convolution of $\Delta F(\tau)$ with the recovery kernel $K_{REC}(t - \tau)$. The effect of aging is included by changing at different stages of the simulation the functions describing $N(\tau)$ and $K_{REC}(t - \tau)$. Hence, a series of simulation runs yields the probabilities of obtaining F = 1 before a given time t_j. At a later stage of the study, a feedback loop from the value of F(t) to the history of the environment can be added. Such a loop would account for facts such as the changes of moisture content of the subgrade due to moisture infiltration through newly formed cracks.

Summary of Cumulative Damage Model

This section presented the framework for a pavement cumulative damage model. This model uses the primary responses as an intermediary step in the process of computing the limiting responses. The primary responses are obtained for linear elastic and viscoelastic layered systems under varying loads and environmental conditions. These primary responses are used to predict three components of the damage: rutting, slope variance, and fatigue cracking. The damage functional assumes three independent mechanisms: a cumulative fatigue process, a healing or recovery process, and an aging process. The model can include some of the nonlinearities of the damage functional in feedback loops, which account for interactions between the output variables (cracks and deflections) and the input variables (loads and environmental variables). Simulation techniques are applied to this model in order to account for the stochasticity of the input variables.

SUMMARY AND CONCLUSIONS

The objective of this review of literature is to identify the modes of damage and their initiation, propagation, and accumulation in the flexible pavement structures. The review is limited to only load-associated damage and its influence on the structural integrity of the pavement. The study is performed by first reviewing the concept of damage in engineering material, thus providing the necessary background work for the development of a damage concept in structure-sensitive engineering systems. Then the pavement system and its modes of damage are reviewed with special emphasis on the mode of damage associated with the repeated loading.

Finally, the framework of a comprehensive model for analysis of damage in highway pavement is presented. This framework consists of a three-layer viscoelastic model and a cumulative damage concept used in conjunction with a simulation technique.

The results of this study substantiate the following conclusions:

1. Pavement failure is a many-sided problem, and it is the result of a series of interacting complex processes, none of which is completely understood. The question of what constitutes the failure is highly subjective and depends on the user's evaluation of the facility.
2. The damage in pavement structures is accumulative and depends on the external excitations, loading and environmental variables, and the physical factors that measure the competence of the system.
3. The input variables and the capabilities of the pavement to resist the initiation and growth of damage can at best be represented in a stochastic manner.
4. Development of any comprehensive model for analysis of damage in a pavement structure should take into account (a) the subjective nature of definition of failure, (b) the cumulative nature of damage, and (c) variabilities present in materials properties, environmental factors, and load.

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SERVICEABILITY PERFORMANCE AND DESIGN CONSIDERATION

W. Ronald Hudson

What is pavement failure? How can pavement failure be defined? How can pavement design be related to quantitative measures of pavement failure? These questions have plagued pavement designers for centuries and are still of concern to us today. Every man involved with pavement design must answer these questions for himself. If a successful, concerted attack is to be made on the problem, however, we must use the same answers or at least compatible answers to these questions.

This problem was illustrated in some detail at the WASHO Road Test, where a panel of experts was involved in establishing a level of failure for the pavement sections being tested. There was a considerable difference of opinion among the experts as to when each section had failed. These problems led Carey and Irick (1) to investigate pavement failure and to define a "pavement serviceability-performance concept" for use at the AASHO Road Test. Acceptance of this concept is not the issue here; rather, our task is to establish a pavement system output function that may be used as the objective function in the systems engineering process for asphalt concrete pavement design.

Previous research has set the stage. The problem has been broken down into logical parts, and papers have been presented at this workshop on material characterization, solutions to boundary value problems, and distress analysis. It is important, however, that we keep in mind the necessity to bridge the gap between these individual effects and the pavement failure.

A crack may be an indicator of the material failure; but it is not the pavement failure. A crack may be undesirable in a pavement (to a certain extent it is not undesirable in continuously reinforced concrete pavements, for example); however, it is not the "failure" of the whole system.

A deflection of 0.25 or 1.0 in. is also not a pavement failure. However, it may be a clue in some cases that the pavement is overloaded and that distress is imminent. Such limiting deflections may have through experience been selected as the design criteria for a particular class of materials in a particular design situation, as in the CBR or other design methods, but these limiting deflections must not be mistaken for pavement failure.

Because the output function is defined in terms of performance and because performance as well as distress mechanisms associated with it have a variety of connotations, a list of definitions is presented to ensure a uniform basis for the ensuing discussion. The definitions are generally based on concepts developed by Carey and Irick (1) for evaluating the performance of the various pavements at the AASHO Road Test and are the same as those used by Hudson et al. (2). It should be noted that inherent in the definitions and the development of the equations for the pavement system is the purpose of the highway facility, which is "to provide a safe, comfortable, and economical method of transporting goods and people."

The following definitions are used in this paper:

1. Performance is a measure of the accumulated service provided by a facility, i.e., the adequacy with which a pavement fulfills its purpose. Performance is often specified by a performance index as suggested by Carey and Irick (1). As such, it is a direct function of the serviceability history of the pavement.
2. Present serviceability is the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic in its existing condition.

(The definition applies to the existing condition, i.e., to the condition on the date of rating, and not to the assumed condition the next day or at any future or past date.)

3. Behavior is the reaction or response of a pavement to load, environment, and other inputs. Such response is usually a function of the mechanical state (i.e., stress, strain, or deflection surface properties), which occurs as a primary response to the input.

4. Distress modes are those responses that lead to some form of distress when carried to a limit; e.g., deflection under load is a mechanism that can lead to fracture. Some behavioral responses may not provide distress mechanisms.

5. Distress manifestations are the visible consequences of various distress mechanisms that usually lead to a reduction in serviceability.

6. Fracture is the state of being broken apart or of the member or material being cleft and includes all types of cracking, spalling, and slippage.

7. Distortion is the state of change of the pavement or pavement component from its original shape or condition. Such changes are permanent or semipermanent as opposed to transient, such as deflections.

8. Disintegration is the state of being decomposed or abraded into constitutive elements (i.e., stripping, raveling, or scaling).

With this brief background and these definitions, let us proceed to look at the problem in some detail, keeping in mind that the design of pavements primarily has a functional overtone. At every step of the pavement design or management process, we must keep in mind the function the pavement is to serve.

SERVICEABILITY REVISITED

The primary operating characteristic of a pavement at any particular time is the level of service it provides to the users. In turn, the variation of serviceability with time is some measure of the pavement performance. This performance and the cost and benefit implications are the primary considerations of design and overall management system.

It is important at this point to differentiate between two types of pavement evaluations. They are both important, and neither is designed to replace the other.

A functional evaluation is typified by the serviceability-performance concept and answers the question, How well is the pavement currently serving its function? This is sometimes called a user-oriented evaluation.

A mechanistic evaluation of the pavement is equally important and is associated with determining the pavement's mechanical condition with the purpose of improving future performance; e.g., a mechanistic evaluation is mainly an indicator of action needed to maintain serviceability and, in that sense, may be a precursor to the serviceability evaluation.

Concept of Serviceability

Many words and methods have been used to describe the concepts of performance and serviceability. One of the best known procedures for defining and obtaining serviceability was established at the AASHO Road Test (1). It was based on subjective evaluation by the road user of the riding quality provided by a pavement at a given (the present) time. To develop the method, the researchers performed correlations with physical measurements of the surface characteristics for a large set of test pavements, and the result was termed the present serviceability index (PSI). This PSI has been extensively used, in its original form and in many modified forms, to predict pavement serviceability. The intergration of PSI over time or over the summation of load applications was termed performance.

Although the serviceability-performance concept represented very real progress and is widely used by many agencies, the ensuing years have seen considerable confusion. This has partially resulted from a proliferation of modifications of the basic method and also from a lack of appreciation and understanding of some of the fundamental considerations of pavement failure. It has further stemmed from a seeming lack of appreciation that, whereas PSI is measured on an objective basis, its purpose is to estimate the subjective opinion of the road users.

Among the purposes of this paper are to define the rationale that underlies pavement performance evaluation, to attempt to clarify some of the concepts underlying serviceability measurements, and to define the role of pavement performance evaluation within an overall pavement management system.

Performance as a Pavement System Output

The process of managing pavements consists of a variety of planning, design, construction, operation, and research activities. Attempts have recently been made by a number of investigators (2 through 8) to define part (including design subsystems) or all of this process in terms of a formal systems framework. These efforts have explicitly recognized that one of the major activities involved is that of performance evaluation or feedback.

If we accept the fact that the currently imperfect state of technology in the pavement field requires such performance evaluation, then we must first define what outputs of the system are to be evaluated. Figure 1 shows the gross output of two alternative pavement strategies in terms of their serviceability-age histories (performance) and the associated value implications. (Pavement strategy includes the structural design, the materials used, the construction processes and control adopted, maintenance procedures, seal coats, and resurfacing.) A large number of traffic, materials, climatic, construction, maintenance, and other variables combine to produce any one such performance profile. These variables are all reflected in the overall pavement strategy that is adopted, and the performance achieved depends on this.

The distinction between serviceability and performance is important. Serviceability (Fig. 1) is a measure only of the pavement's ability to serve its function at a particular time (i.e., at the present). The past record or suspected future capacity of the pavement is not considered in a single PSI measure. Performance is the history of these single PSI measures (called serviceability-age history) of the pavement. Age rather than equivalent wheel load carried (EWL) is taken as the primary abscissa in Figure 1 in order that value implications can be taken into account. Furthermore, it is not sufficient to know or predict only the initial

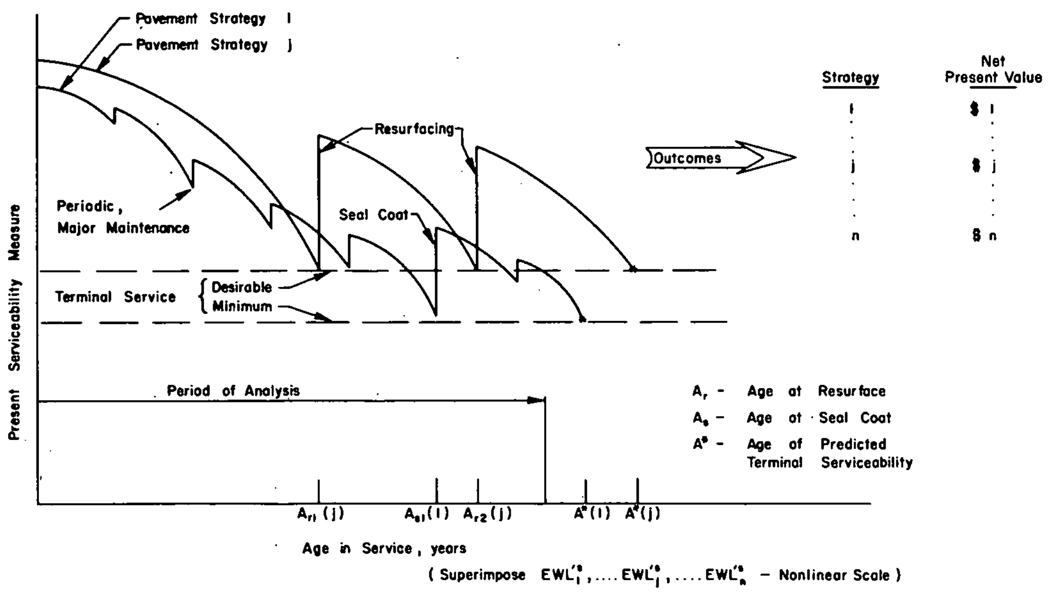

Figure 1. Gross output of a pavement system in terms of performance and value implications.

serviceability or the terminal age. Without knowing the intermediate portion, we cannot adequately check the design strategies, their plans, or programs for maintenance and resurfacing; nor can we explore the implications of raising the terminal serviceability level.

Role of Performance Evaluation in the Pavement Management System

The measurement of the outputs of a pavement system during its time in service, i.e., the evaluation of its performance, has previously been noted as a major management activity. Figure 2 shows the principal elements of this activity as a portion of the overall pavement management system and the information flows that result in a continuous process of feedback. The development and implementation of the performance evaluation subsystem as a portion of the management system or of its components can be a comprehensive and major systems problem within itself. Several aspects of this are subsequently discussed in more detail.

EVALUATING PAVEMENT DISTRESS

Distress is normally evaluated by the two basic approaches: the functional evaluation of the effect of distress on the function of the roadway (i.e., how well it is serving traffic today) and the mechanistic evaluation of distress with an eye toward future performance (i.e., what the current physical condition of the pavement is, what its causes are, and what effect this condition will have on the future performance of the pavement).

The difference between these two approaches is the key to the problem of relating pavement behavior to pavement performance. A crack in the pavement surface may have minor or no effect at all on how well the pavement is serving traffic today. On the other hand, the maintenance engineer and the design engineer, who look at this existing crack in terms of mechanistic evaluation, immediately think of it as a local failure. The design engineer may be concerned if he did not expect the crack to occur. The mainte-

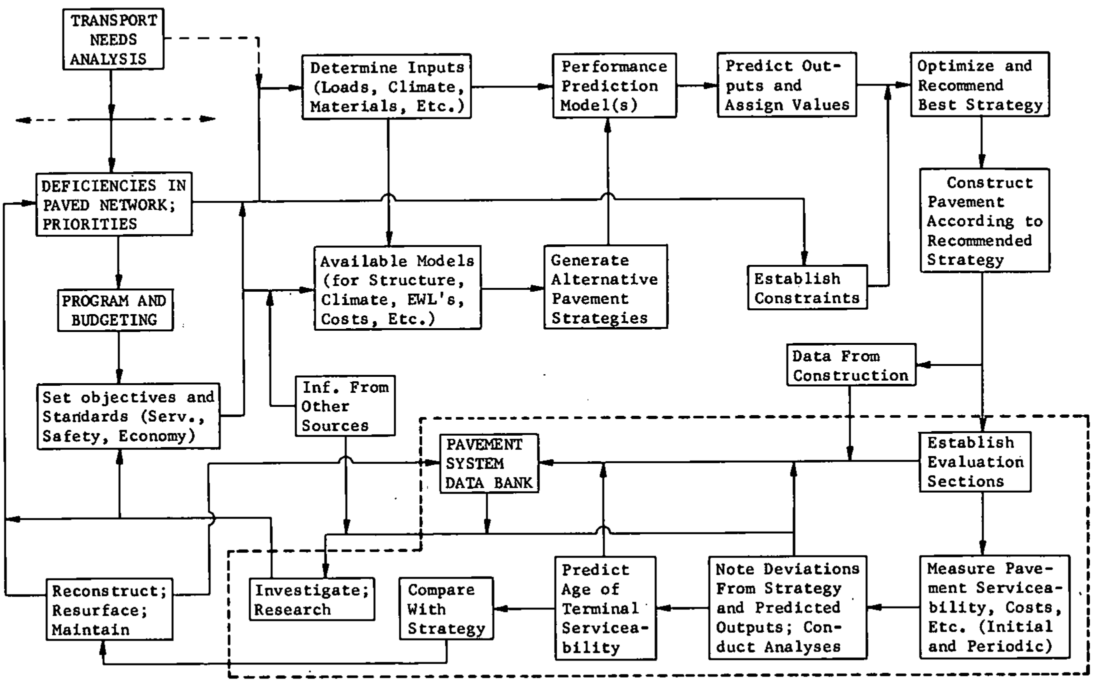

Figure 2. Role of performance evaluation in a generalized pavement management system.

nance engineer may be concerned if he thinks the crack will permit intrusion of water, increased deflection, spalling, and additional cracking that may occur and can result in rapid deterioration of the pavement. If on the other hand the pavement is designed to contain cracks, their presence will be of no concern to the designers or the pavement users involved.

However, if the roughness of the pavement, either as constructed or induced by changes in the pavement surface profile, is undesirable in character, i.e., excites poor response from the pavement user or is excessive in nature and provides an undesirable ride, the designer and the maintenance engineer may not be concerned at first, and yet the pavement will be a poor one.

OBTAINING A SYSTEM OUTPUT FUNCTION

Figure 3, as presented by Hudson et al. (2), shows that the expected output of a structural systems model is a behavioral characteristic, deflection or strain, that results at some limiting value in distress. The terms rupture, distortion, and disintegration have been used to describe all types of distress. The figure indicates that these are combined with appropriate weighting functions to yield a wear-out curve or system output function for the pavement. It is no easy task to develop a method of relating these factors.

A research team from Texas (10) has developed a working pavement design system that accomplishes this purpose, as shown in Figure 4. In effect, the team simplified the problem by using a deflection model for relating inputs to output—in this case surface deflection under the load. As indicated in the figure, the tie between expected deflection and expected performance was made empirically from equations developed at the AASHO Road Test relating deflections to performance. Such empirical methods are often used to bridge the gap between predictions of behavior and expected performance. Other researchers (11) have bridged this gap by using the AASHO Road Test

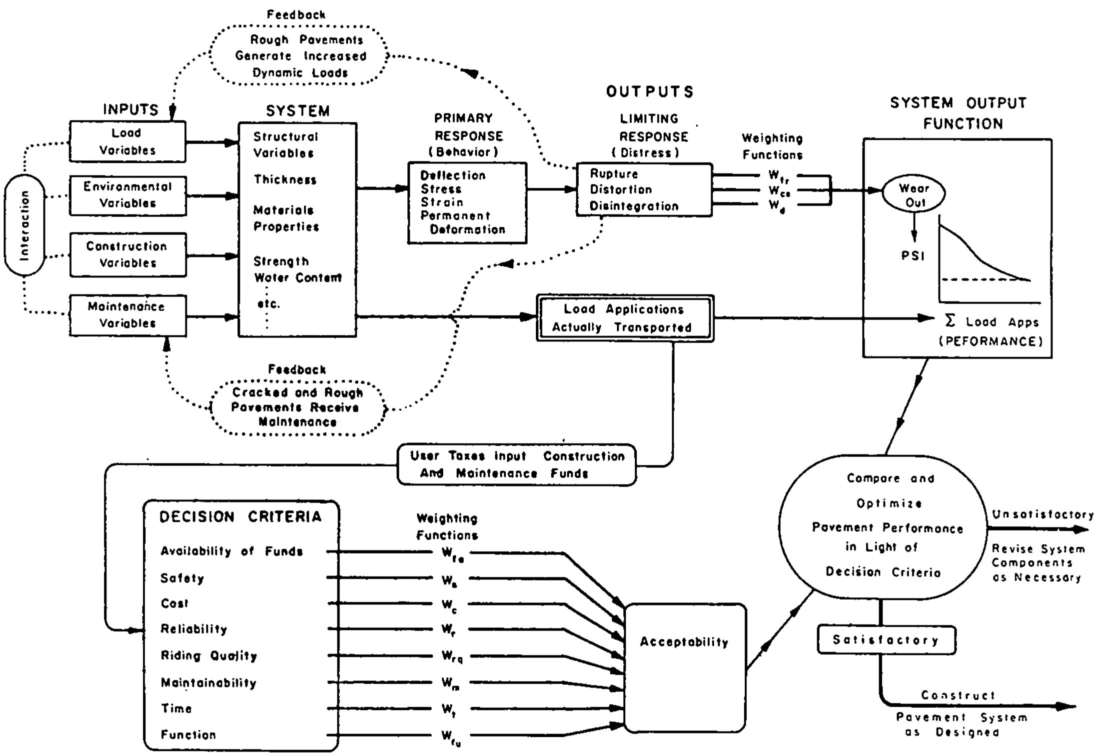

Figure 3. Ideal pavement system.

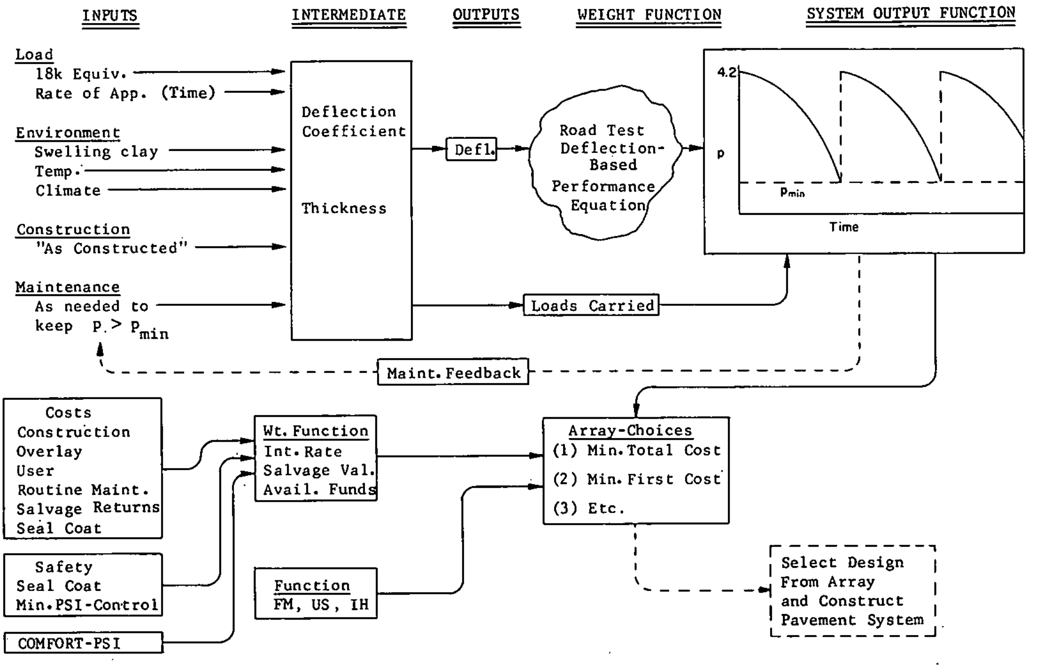

Figure 4. Working pavement system.

structural number concept to empirically predict performance from material properties in layers and their thicknesses. In both cases, the effect of system environments, in these cases the effect of subgrade swelling clay on pavement roughness, is evaluated and integrated into the serviceability-deterioration function by empirical evidence obtained in a study of Texas pavements.

RELATIONSHIP BETWEEN BEHAVIOR AND PERFORMANCE

Examination of the conceptual pavement system shown in Figure 3 illustrates the complex relationship that exists among the following:

1. System inputs,
2. Materials composing the system,
3. Pavement behavior, and
4. Pavement performance.

It is necessary to compare the performance or system output function against various decision criteria in order to make rational pavement designs and management decisions. It will ultimately be necessary to do this comparison on a stochastic basis; however, at the present time, it is adequate to make the comparison deterministically to illustrate the concept.

Hudson et al. (2) has attempted to associate material properties with modes of failure or distress through considerations of the various mechanisms and manifestations of distress. Limiting response (i.e., distress) modes have been divided into three categories: rapture or fracture, distortion, and disintegration. With the exception of pavement slipperiness associated with the surface coefficient of friction, all forms of pavement distress can be related individually or collectively to these modes.

The next logical step might be to list the pertinent material properties for each of the failure mechanisms associated with each manifestation of failure; however, this is beyond the scope of this paper.

SUMMARY

The concepts of performance and the solution to pavement design are complex problems. The application of systems engineering techniques appears to offer a reasonable approach to the solution. In this process, certainly materials characterization, theoretical analogies, solutions to boundary value problems, and distress analyses are important aspects of the problem. However, it is essential to pavement design that consideration be given to the functional requirements of the pavement and that pavement failure be defined in terms of the function of the pavement and not merely in terms that are convenient to analyze and predict from some mechanisms of failure.

At the present time, most theories associated with pavements predict pavement behavior and pavement distress. Available work on pavement performance involves primarily empirical relationships between measurements on pavements and observed serviceability. The most complete example of this type involves use of the AASHO Road Test data.

It is recommended that, as research effort continues toward the development of a better pavement design method, adequate attention be given to combining various pavement behavior and distress factors into an overall performance function because it is only through adequate definition of this function that the pavement problem will ultimately be solved.

Generalized Failure Concept

To accomplish the design of the pavement system requires that a definition of failure be fully specified. The term "failure" as used here refers to a failure of the pavement system and not the material failure. A key point in this discussion is that failure of a pavement material is generally not a catastrophic occurrence, as is the case of a steel rod rupturing in tension. Failure of pavement is, instead, a condition that develops gradually over a span of time generally measured in years. In this framework, the output of the pavement system exceeds some limiting value formulated by the decision criteria. A pavement designated as having "failed" in some respect may still be capable of carrying traffic at a reduced service level and may still have a high salvage value in an economic analysis for a pavement rehabilitation program.

The conceptual pavement system (Fig. 3) provides the framework for development of a generalized model of pavement failure. Figure 3 shows that the pavement system output and the decision criteria should be considered together because the decision criteria are used to evaluate the system output and to make a judgment of pavement performance. Thus, failure may be defined by the decision criteria as some limiting value of the system output.

Distress Index

The behavior of a pavement structure may be quantified in terms of its response. Figure 3 also shows that the limiting response is known as distress (i.e., rupture, distortion, or disintegration) and may be expressed conceptually as

$$\underline{DI}(\underline{x}, t) = \underset{s=o}{\overset{s=t}{\underline{F}}} \left[\underline{C}(\underline{x}, s),\ \underline{S}(\underline{x}, s),\ \underline{D}(\underline{x}, s)\underline{x},\ t\right] \tag{1}$$

where

t = time
$\underline{x}$ = a space variable;
$\underline{DI}(\underline{x}, t)$ = distress index, a function of space and time;
$\underline{C}(\underline{x}, t)$ = measure of fracture, a function of space and time;
$\underline{S}(\underline{x}, t)$ = measure of distortion, a function of space and time; and
$\underline{D}(\underline{x}, t)$ = measure of disintegration, a function of space and time.

Distress is spatial in nature and is best considered on a unit volume basis. The notation in Eq. 1 indicates that the distress index is a function of the history of the variables shown from time zero to current time t.

Each of the parameters in Eq. 1 must be quantitatively predicted from the input parameters and the system models. Considering the systems framework, we may express rupture, distortion, and disintegration in general as functions of five classes of variables. For rupture, $\underline{C}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural variables and of space and time. For distortion, $\underline{S}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural variables and of space and time. For disintegration, $\underline{D}(\underline{x}, t)$ is a function of load, environment, construction, maintenance, and structural variables and of space and time.

These expressions predict the three modes of distress in terms of five classes of variables. The variables are all expressed as a function of space and time with one exception. Construction variables enter at the beginning of the time history. After a pavement is constructed and opened to traffic, it is no longer time-dependent on the methods of construction.

The next development step is to substitute these expressions into Eq. 1, which conceptually describes the upper region of Figure 3. When the proper weighting functions are used, Eq. 1 would represent the system output function, which may then be evaluated in terms of various decision criteria.

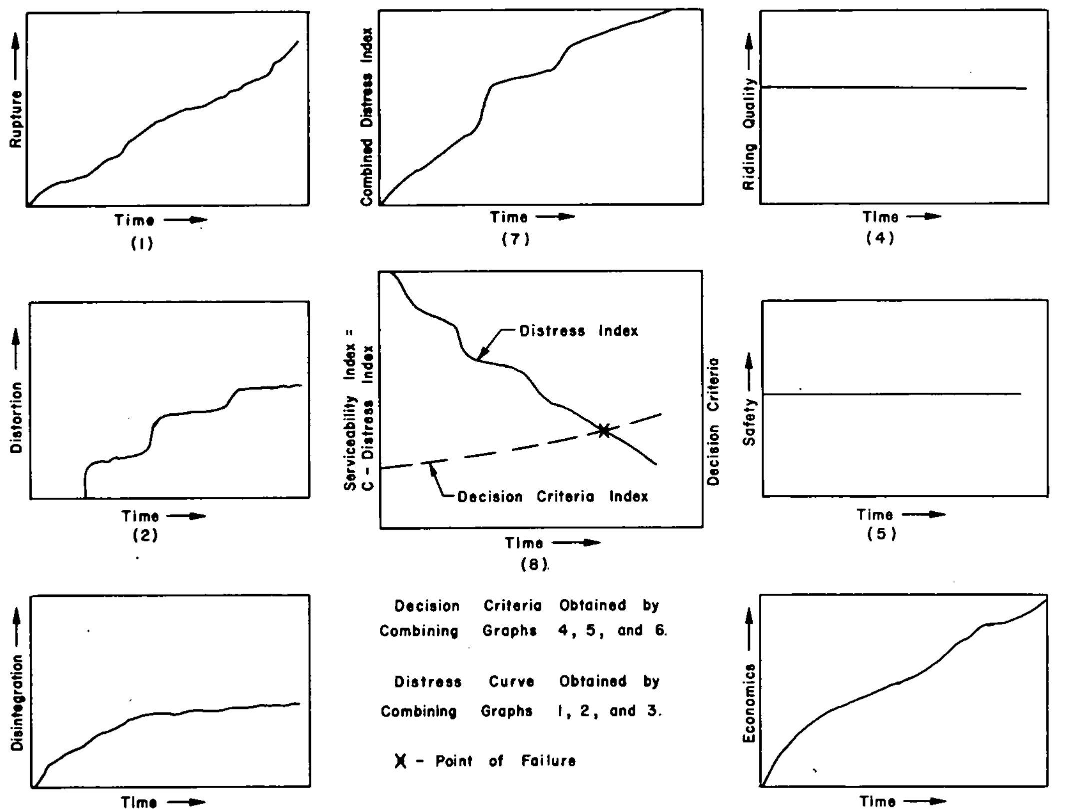

Figure 5. Failure concept.

Decision Criteria Index

Basically, an engineer's criterion for judging a pavement structure is how well it is accomplishing its purpose. The decision criteria should therefore include, among other things, riding quality, economics, and safety. These decision criteria may be expressed in terms of a decision criteria index, or $\underline{\text{DCI}}(x, t)$, as $\underline{\text{DCI}}(\underline{x}, t)$ is a function of riding quality, economics, safety, maintainability, and other factors and of space and time.

All the parameters included in the decision criteria index are functions of space. The time term is not included in riding quality because there is a minimum allowable rideability for any given type of roadway regardless of time. The safety term is also time-invariant because it, too, has some minimum acceptable level, for given conditions, that should not be exceeded during the life of a pavement. Because a highway represents a capital investment that may be depreciated over some time period, there is need for considering time in the economics term.

Each of the parameters in this expression must be quantified. Thus far, there has been little attempt to do so. Generally, these factors are considered subjectively, either directly or indirectly, by highway administrators.

System Failure

Failure of the pavement structural system may be expressed as a condition where the distress from the system output has exceeded an acceptable level based on the decision criteria. Figure 5 shows the principles of this failure definition. Through a model similar to that shown in Eq. 1, rupture, distortion, and disintegration may be combined into a distress index as shown by the solid line in graph 7 of Figure 5. The serviceability curve is then some constant minus distress function depending on the scaling factors involved.

An illustration of what acceptable levels might be for each decision criterion is presented to the right in Figure 5. The decision criteria are represented by a combined function expressed by the dashed line in graph 8. The point at which these two curves intersect might then represent failure for the system, i. e., when the pavement is performing at less than the desired level. Other functions of the variables such as the area between the curves may also be appropriate measures of the performance.

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MATERIALS CHARACTERIZATION—EXPERIMENTAL BEHAVIOR

John A. Deacon

Proper structural design of asphalt concrete pavement systems relies in part on a thorough understanding of the response of the constituent materials to load. Such response has two components: (a) strength, which represents the limiting condition such as fracture or slip, and (b) deformability, which represents the stress-strain-time response before the failure or limiting condition is attained.

Even though a great deal of research has been devoted to characterizing the response of these materials, there is a noticeable lack of accord as to proper test procedures, theories of behavior, and test results. This situation is readily explainable in view of the following:

1. The variety of materials encountered by the pavement designer is unlimited because of the nature of these multiphase materials and the manner in which they are manufactured from, in large part, locally occurring but often artificially processed ingredients;
2. The nature of the pavement structure in which these materials are used depends greatly on the function to be performed by the pavement and varies, for example, from an oil treatment of an unprepared soil to a substantial thickness of asphalt concrete placed on a high-quality treated base course over a carefully prepared subgrade;
3. During the service life of a pavement, a number of environmental conditions change including temperature and moisture content, and the material properties are altered because of factors such as thixotropy, aging, curing, and densification;
4. The response of pavement materials to loading is extremely complex and for the most part even under in-service stress intensities is characterized by nonlinear, inelastic, rate-dependent, anisotropic, and sometimes temperature- and moisture-sensitive behavior;
5. Until recent years, solutions to pertinent boundary value problems have been nonexistent or at least not readily available; and
6. The approach to the problem has been piecemeal at best and has involved many different researchers from many different agencies each striving for an optimal solution to a singular problem of limited scope and sometimes prejudiced intent.

The purpose of this paper is to summarize some of the experimental laboratory work that has been devoted to characterizing the behavior of pavement materials on a phenomenological basis. Attention is limited to cohesive and cohesionless soils, unbound aggregate bases, and bituminous paving mixtures and is concentrated on those test procedures that seem capable of providing the most useful results for rational pavement analysis. Pertinent variables affecting material response, an understanding of which is essential for proper interpretation of test results, is given in the following outline.

- I. Loading variables
  - A. Stress history (nature of prior loading)
    - 1. Nonrepetitive loading (such as preconsolidation)
    - 2. Repetitive loading
      - a. Nature, whether simple or compound
      - b. Number of repetitive applications
  - B. Initial stress state (magnitude and direction of normal and shear stresses)

C. Incremental loading
    1. Mode of loading
        a. Controlled stress (or load)
        b. Controlled strain (or deformation)
        c. Intermediate modes
    2. Intensity (magnitude and direction of incremental normal and shear stresses)
    3. Stress path (relation among stresses, both normal and shear, as test progresses)
    4. Time path
        a. Static
            (1) Constant rate of stress (or load)
            (2) Constant rate of strain (or deformation)
            (3) Creep
            (4) Relaxation
        b. Dynamic
            (1) Impact
            (2) Resonance
            (3) Other, including sinusoidal (rate of loading is variable) and pulsating (duration, frequency, and shape of load curve are variables)
    5. Type of behavior observed
        a. Strength (limiting stresses and strains)
        b. Deformability
    6. Homogeneity of stresses
    7. Drainage

II. Mixture variables
    A. Mineral particles
        1. Maximum and minimum size
        2. Gradation
        3. Shape
        4. Surface texture
        5. Angularity
        6. Mineralogy
        7. Adsorbed ions
        8. Quantity
    B. Binder
        1. Type
        2. Hardness
        3. Quantity
    C. Water (quantity)
    D. Voids
        1. Quantity
        2. Size
        3. Shape
    E. Construction process
        1. Density
        2. Structure
        3. Degree of anisotropy
        4. Temperature
    F. Homogeneity

III. Environmental variables
    A. Temperature
    B. Moisture
    C. Alteration of material properties with time
        1. Thixotropy
        2. Aging
        3. Curing
        4. Densification

Test configurations are listed in the following outline.

- I. Tension
  - A. Uniaxial tension
  - B. Indirect (splitting) tension
  - C. Cohesiometer
- II. Compression
  - A. Unconfined, uniaxial compression
  - B. Triaxial compression
    - 1. Open system
      - a. Isotropic compression
      - b. Conventional triaxial compression, whether normal, vacuum, or high-pressure
      - c. Box with cubical specimen
    - 2. Closed system
      - a. Oedometer
      - b. Cell
      - c. Hveem stabilometer
- III. Flexure
  - A. Rotation
    - 1. Roating
    - 2. Nonrotating
  - B. Loading
    - 1. Cantilever
    - 2. Simple beam
      - a. Point support
      - b. Uniform support
- IV. Direct shear
  - A. Direct shear (rigid split box)
  - B. Double direct shear
  - C. Uniform direct shear (rigid caps with confined rubber membrane and split rings for lateral restraint)
  - D. Uniform strain direct-shear (hinged box)
  - E. Punching shear
- V. Torsion
  - A. Pure torsion
  - B. Triaxial torsion
  - C. Specimen shape
    - 1. Solid cylinder
    - 2. Thick-walled, hollow cylinder
- VI. Indirect
  - A. Penetration tests
  - B. Squeeze tests
  - C. Marshall stability
  - D. Angle of repose
  - E. Others

Possible specimen shapes are enumerated in the following.

- I. Rectangular parallelepiped
  - A. Short
  - B. Long
  - C. Cubic
- II. Cylinder
  - A. Solid
    - 1. Short
    - 2. Long
  - B. Thick-walled, hollow
    - 1. Short
    - 2. Long
- III. Plate
- IV. Other

STRENGTH

Strength represents the limiting or failure response of materials to load. In general, pavement materials can fail in one of three ways: (a) fracture due to excessive tensile loads (induced primarily by traffic or thermal gradients), (b) fracture due to repetitively applied tensile loads having magnitudes less than the ultimate tensile strength (fatigue), and (c) slip or relative displacement due to the action of shearing stresses. (The manner in which pavements fail or become distressed is discussed in other papers included in this Special Report.) Crushing failures involving individual aggregate particles are of no significance in pavement systems.

Tensile Strength

Only those bound components of the pavement structure are capable of withstanding significant tensile stresses without rupturing. Of these, only the asphalt paving mixture is considered here. Tensile strength of asphalt mixtures is considered important in three areas of design, including (a) fracture due to the single application of a large, normally applied load; (b) slippage, such as might be induced by large braking forces; and (c) thermal cracking.

Kennedy and Hudson (55) have reviewed tensile testing equipment and procedures that have been applied to highway materials including the direct tensile test (specimen usually cemented by an epoxy resin to end caps), beam tests (including simply supported beams and the cohesiometer), and indirect or splitting tensile tests. They concluded that the indirect test was the most appropriate for examining tensile properties of highway materials. Regardless of the test equipment used, the tensile strength is usually defined as either the peak stress on the resulting stress-strain curve or the stress at break.

Table 1 gives a number of variables that are known to influence the tensile strength of asphalt mixtures. Most notable among these variables are probably rate of loading and temperature except at low temperatures, where rate of loading and temperature increments have minor effects. Tensile strengths at low temperatures have been reported to range from about 500 to 1,400 psi (24).

Of interest also is the strain or elongation that occurs at the point of rupture. Monismith and colleagues (78) suggest that an appropriate means for examining this strain is a double logarithmic plot of rupture stress (factored by the ratio of 293 to the absolute temperature) versus strain at rupture. Their data show that a maximum rupture strain of about 1 percent occurs at an intermediate rupture stress level and that minimum rupture strains of about 0.1 percent occur at both larger and smaller rupture stress levels. For the conditions investigated by Haas and Anderson (33), the fracture strain decreased monotonically as fracture stress increased. In any case it appears that the minimum fracture strain occurs at low temperatures and is on the order of 0.1 percent (24). Table 2 gives some of the variables influencing the tensile strain at rupture.

Finally, it may be possible to relate tensile strength to mixture stiffness (a measure of deformability). Haas and Anderson (33) show that there can exist an optimum stiffness modulus that yields a maximum tensile strength.

Fatigue Strength

The repetitive application of tensile stresses having magnitudes less than the tensile strength can ultimately cause fatigue cracking in bound materials. Attention is limited here to a review of the fatigue behavior of asphalt paving mixtures.

In recent years, a number of laboratory investigations have been initiated to define the fatigue behavior of asphalt paving mixtures, to determine the causes of observed fatigue behavior, and to ascertain the influence of mixture variables on this behavior. These investigations have used a variety of specially built equipment. The range of equipment capabilities and test configurations is indicated as follows:

1. Mode of loading—controlled stress, controlled strain, and intermediate but undefined modes;

TABLE 1

TENSILE STRENGTH OF ASPHALT MIXTURES

Variable	Change in Variable	Effect on Tensile Strength	Reference	Remarks
Loading				
Rate of loading	Increase	Increase	21, 40, 42, 47, 55, 119	Effect increases as temperature increases
Type of test	Change	Change	53	Strength significantly greater for splitting tests than direct tests
Mixture				
Asphalt source	Change	Change	33	
Asphalt content	Increase	Increase	34	3.5 to 7.0 percent
	Increase	Optimum	40	
Asphalt hardness	Increase	Increase	34, 47	
	Change	Change	42	50 to 110 penetration; possible maximum
Compaction temperature	Increase	Increase	34	200 to 300 F
Mixing temperature	Increase	Increase	34	250 to 350 F
Type of compaction	Change	Change	34	Stronger for impact than for gyratory shear
Aggregate type	Change	Change	34	Stronger for crushed limestone than for rounded gravel
Mineral filler type	Change	Change	37	Asbestos mineral filler produced considerably higher strength at low temperatures
Filler-bitumen ratio	Increase	Optimum	20	
Environment				
Temperature	Decrease	Increase	21, 33, 40, 47, 53, 119, 125	Particularly significant in range of 20 to 80 F; relatively unaffected at low temperatures

2. Specimen shape—cylindrical, beam, and circular with reduced cross section, plate, and trapezoidal beam;
3. State of stress—uniaxial, biaxial, and triaxial;
4. Load type—sinusoidal and pulsating;
5. Load frequency—3 to 3,000 cycles per min; and
6. Specimen support—rigid, spring, and pressurized fluid.

TABLE 2

TENSILE RUPTURE STRAINS OF ASPHALT MIXTURES

Variable	Change in Variable	Effect on Rupture Strain	Reference	Remarks
Loading				
Rate of loading			119	No clear effect
Mixture				
Asphalt content	Increase	Increase	24, 119	
Filler-bitumen ratio	Increase	Optimum	20	
Asphalt hardness	Increase	Decrease	42	
Environmental				
Temperature	Increase	Increase	33, 42, 119	

Each particular apparatus has its special advantages and, of course, its limitations. None is sufficiently universal to warrant its adoption as a standard testing device, and all must be considered as research tools having specific and limited uses. Because of this, extreme care must be exercised for the valid interpretation of test data and for their use in pavement design.

The following represent the current state of knowledge of fatigue of asphalt paving mixtures.

1. Under conditions of constant air void and asphalt contents, the determinant of fatigue failure is the maximum principal tensile strain in the mixture (90).
2. Mixture stiffness plays a dominant role in determining fatigue behavior, and any factor influencing mixture stiffness may likewise affect fatigue behavior.
3. Damaging fatigue loading causes a reduction in flexural strength (modulus of rupture) (77). Mixture stiffness is likewise decreased by fatigue loading, but the magnitude of the decrease is a function of the method for measuring stiffness (6, 14, 90).
4. Mode of loading has a tremendous effect on fatigue behavior. For identical initial stresses and strains, the fatigue life (number of repetitions to failure) is considerably larger for controlled-strain loading than for controlled-stress loading (75).
5. The roles of mixture stiffness and mode of loading are interrelated. For controlled-stress loading, specimens exhibiting the largest initial stiffnesses tend to perform most satisfactorily (largest fatigue life at a given stress level) as long as the mixture is nonbrittle and has a reasonable balance among the proportions of its constituent materials. The reverse appears to be true for controlled-strain loading (75).
6. For controlled-stress loading, the mean fatigue life, $\overline{N}$, is related to the applied stress level, σ, as follows:

$$\overline{N} = K_1(1/\sigma)^{C_1} \tag{1}$$

where K_1 and C_1 are constants. There is no evidence of an endurance limit (a limiting value of stress below which a material can endure an infinite number of stress cycles without failure) up to at least 10^8 load applications (15, 52, 90). If the mixture is linear, then the following equation is also valid

$$\overline{N} = K_2(1/\epsilon)^{C_2} \tag{2}$$

where K_2 and C_2 are constants and ϵ is the initial tensile strain. Similar equations can be used to define the simple-loading fatigue behavior under controlled-strain loading; however, the possible existence of an endurance limit under such loading is unknown (75).

7. In evaluating variables whose effects on fatigue behavior can be explained primarily in terms of associated effects on mixture stiffness, all data obtained under controlled-stress loading can often be represented by a single equation, Eq. 2. Thus, Pell and Taylor (93) found it possible to express the effects of temperature and speed of loading in the single form of Eq. 2, and Bazin and Saunier (6) and Kirk (56) used a similar procedure to evaluate the effect of temperature.
8. The constants of Eqs. 1 and 2 depend on mixture composition, conditions of testing, and the definition of failure. Recently reported values for the exponent C_1 in the controlled-stress mode of loading include 2.5 to 5.9 for an asphalt concrete depending on asphalt type (96); 5.3 to 5.9 for linear mixtures and 1.6 to 3.5 for nonlinear mixes depending on asphalt penetration (93); and 5.2 for a sandsheet mixture (6).
9. The linear summation of cycle ratios governs fatigue behavior of asphalt mixtures that are subjected to multiple strains of varying and random magnitude (15). This means that at failure

$$\sum_i (n_i/N_i) = 1 \tag{3}$$

where n_i is the actual number of applications of strain ϵ_i under the compound loading and N_i is the number of applications of strain ϵ_i that would have resulted in failure under simple loading in which ϵ_i was repetitively applied by itself.

10. Fatigue test data exhibit extreme variability as compared with other testing methods. The fatigue life of specimens tested in simple, controlled-stress loading under supposedly identical testing conditions can be approximated by the logarithmic normal probability distribution (93).

11. On the basis of limited evidence it is possible that rest periods may be beneficial for asphalt paving mixtures depending on the length of the period, the temperature, the characteristics of the mixtures, and the stress conditions existing within the mixture (6, 90). Peel (93) recently concluded that possible beneficial effects will be observed only for mixes made with very soft binders and rested at high temperatures under a compressive stress.

12. A list of factors known to influence fatigue behavior of asphalt mixtures is given in Tables 3 and 4. Monismith and Deacon (75) give explanations of most of these effects.

13. It has recently been proposed that the effects of asphalt viscosity, temperature, mode of loading, and presumably rate of loading for simple forms of testing could be expressed as

$$N_1 = K\, N_0\, (\eta_0/\eta_1)^{\alpha}\, (1/\epsilon)^{c_1} \tag{4}$$

where N_1 is fatigue life at temperature T_1; K is a constant depending on mix; N_0 is an experimentally determined fatigue life at some convenient, standard temperature, T_0; ϵ is dynamic tensile strain in mix; η_0 is asphalt viscosity at T_0; η_1 is asphalt viscosity at T_1; α varies between +1 (controlled strain) and -1 (controlled stress) depending on mode of loading; and n varies between 4 and 6 depending on the type of fatigue test used (99). Extensive future work is warranted to examine the validity of Eq. 4 and to develop it into a working design tool.

Shear Strength

Shear failures can occur in one or more pavement layers if the imposed loads are sufficiently large that the shear stresses exceed the shear strengths. Such failures are perhaps the most catastrophic of all pavement failures inasmuch as large relative dis-

TABLE 3

FATIGUE BEHAVIOR OF ASPHALT MIXTURES FOR CONTROLLED-STRESS LOADING

Variable	Change in Variable	Fatigue Life at Given Stress	Reference	Remarks
Loading				
Rate of loading	Increase	Increase	90, 93	Stiffness increased for loading conditions used
	Increase	Decrease	14	Stiffness decreased for loading conditions used
	Increase	Constant	77	Stiffness constant for loading conditions used
Load duration	Increase	Decrease	14	For pulsating loads
Mixture				
Asphalt content	Increase	Optimum	19, 51, 93	Optimum asphalt content depends on aggregate type and gradation
Air void content	Increase	Decrease	6, 14, 19, 52, 95	
Air void structure			19	Important
Asphalt type			19	Effects primarily mixture stiffness
Asphalt hardness	Increase	Increase	6, 19, 51, 72	
Aggregate type			6, 19, 51	Effect of surface texture and shape small at same asphalt content
Aggregate gradation	Increase	Increase	19, 75	Open to dense-graded
			93	No great effect
Mineral filler	Increase	Optimum	93	
Environmental				
Temperature	Increase	Decrease	75, 93	

TABLE 4

FATIGUE BEHAVIOR OF ASPHALT MIXTURES FOR CONTROLLED-STRAIN LOADING

Variable	Change in Variable	Fatigue Life at Given Strain	Reference	Remarks
Loading				
Strain reversal			77	No effects observed, maximum strain governs
			56	Effects observed, difference in maximum and minimum strains governs
Mixture				
Asphalt content	Increase	Increase	90	Mixture stiffness decreased with increasing asphalt content
Asphalt hardness	Increase	Decrease	75	
			56	No effect observed
Asphalt type	Change	Change	75	
	Change	Change	96	Asphalts yielding stiffer mixes result in reduced fatigue life
Air void content	Increase	Decrease	96	
Aggregate gradation and type			56	Negligible effect
Environmental				
Temperature	Increase	Increase	90, 96	

placements can occur along the slip surfaces. McLeod (67, 68, 69) and Hewit (44, 45, 46, 47) are among those having investigated this type of failure in a fundamental way.

Shear strength is defined as the peak or ultimate stress of a material in shear. It may be measured by direct shear tests or numerous other tests including the triaxial compression test (62, 120). The shear strength is normally interpreted by means of the Mohr-Coulomb failure law, which considers the shear strength to be composed of two parts, one of which is a frictional component that is proportional to the normal stress on the shear surface and the other of which is a cohesive component that is independent of normal stress. Stated mathematically, the Mohr-Coulomb failure law is

$$\tau_f = c + \sigma_f \tan \phi \tag{5}$$

where

τ_f = shearing strength,
σ_f = normal stress on the failure surface at failure,
c = cohesion, and
ϕ = friction angle or angle of shearing resistance.

Although other failure theories have been proposed, it seems likely that the Mohr-Coulomb theory will continue to be effectively used for some time.

The shear strength can be calculated directly from the results of direct shear tests. The shear strength in a triaxial compression test is $(\sigma_1 - \tau_3)_f/2$, where the subscript f refers to the failure condition along a shear plane. This strength is related to c and ϕ as follows (68):

$$(\sigma_1 - \sigma_3)_f/2 = \frac{\sigma_3 \sin \phi}{1 - \sin \phi} + 2\,c\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \tag{6}$$

For the unconfined compression test, the best estimate of the shear strength is $(\sigma_1)_f/2$, which is related to c and ϕ as follows (68):

$$(\sigma_1)_f/2 = 2\,c\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \tag{7}$$

The properties c and ϕ can, for some materials, be evaluated from tension and unconfined compression tests (40). They can also be evaluated from Hveem stabilometer results (89), though perhaps erroneously (40), and from direct shear tests. They are normally evaluated, however, from triaxial compression tests in which a number of identical specimens are tested at different confining pressures, and observations are made of stresses developed at the peak of the stress-strain curves. Mohr circles are constructed to represent the states of stress at failure, and the Mohr failure envelope is drawn tangent to the Mohr circles. The Mohr envelope is generally curvilinear but is usually approximated by the linear relationship of Eq. 5. The properties c and ϕ are determined graphically from the linear approximation of the failure envelope.

Cohesive and Cohesionless Soils—Lambe and Whitman (63) have presented an extensive review of the shear strengths of both cohesive and cohesionless soils. Under drained test conditions and by using effective stress principles, the behavior of these materials can be summarized as follows:

1. In the ultimate condition, [that is, very large shearing strains in which any effects of overconsolidation are minimized] achieved after considerable shearing strain the strength behavior of soil is that of a frictional material. That is, the failure law is

$$\tau_{ff} = \bar{\sigma}_{ff} \tan \bar{\phi}_{ult}$$

The ultimate friction angle $\bar{\phi}_{ult}$ is related to the clay content of the soil. The angle is greatest (about 30°) in pure sand and least (as low as 3 or 4°) in pure clay.

2. At the point of peak resistance, the strength of a normally consolidated soil is also given by a frictional type of failure law,

$$\tau_{ff} = \bar{\sigma}_{ff} \tan \bar{\phi}$$

The angle $\bar{\phi}$ is related to the clay content of the soil. For loose sands $\bar{\phi}$ and $\bar{\phi}_{ult}$ are equal. As the clay content increases $\bar{\phi}$ exceeds $\bar{\phi}_{ult}$ since at the peak resistance the clay platelets within the failure zone have not yet reached a fully oriented, face-to-face alignment.

3. Densification increases the peak strength of soils. For soils with a significant clay content, large stresses suffice to produce an overconsolidated soil, while stresses alone do not effectively densify predominantly granular soils and cycles of loading and unloading are necessary. The failure envelope for densified soils generally is curved, but for practical calculations the peak strength can be represented by a linear relation,

$$\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi}$$

As $\bar{\sigma}_{ff}$ increases, $\bar{c}$ increases and $\bar{\phi}$ decreases.

4. Any change in effective stress changes the density at failure as well as changing the shear strength. Conversely, any action that changes the density at failure must produce a change in shear strength. For the ultimate condition, there is a unique relationship among effective stress $[(\bar{\sigma}_1 + \bar{\sigma}_3)_f/2]$, shear strength $[(\bar{\sigma}_1 - \bar{\sigma}_3)_f/2]$, and water content, such that knowledge of any of these three quantities specifies the other two quantities. [For saturated soils, this unique relationship continues to hold for all types of loading and drainage conditions.] At the peak resistance, this three way relation is not unique . . . .

5. Capillary tensions must be taken into account when determining the effective stresses within soils located above the water table. Because large capillary tensions are possible in clayey soils, such soils can exhibit a large apparent cohesion even though they produce little or no cohesion intercept $\bar{c}$. This apparent cohesion is fully explained by effective stresses.

For saturated soils, the undrained strength may either exceed or be less than the drained strength depending on the type of loading and the degree of overconsolidation. However, differing values of strength (undrained versus drained) can be explained by differences in effective stresses. The undrained shear strength increases with decreasing moisture content, increasing consolidation stress, and increasing maximum past consolidation stress (63).

Tables 5 and 6 give the effects of selected sets of variables on the shear strengths of cohesive and cohesionless soils respectively.

TABLE 5

SHEAR STRENGTH OF COHESIVE SOILS

Variable	Change in Variable	Effect on Strength	Reference	Remarks
Loading				
Rate of loading	Increase	Decrease	122	Drained tests of saturated soils
	Increase	Increase	122	Undrained tests of saturated soils
Overconsolidation	Increase	Increase	63, 122	
Stress path			63	No effect for drained triaxial tests
Effective stress at failure = $(\bar{\sigma}_1 + \bar{\sigma}_3)_f/2$	Increase	Increase	63	Drained triaxial tests
Intermediate principal stress			63	Small effect on drained strength
Mixture				
Compaction energy	Increase	Increase	85	Impact compaction at constant molding moisture content
Adsorbed ions			122	Nature of ions affects strength depending on moisture content
Remolding	Increase	Decrease	122	
Plasticity	Increase	Decrease in ϕ	63	Drained tests based on effective stresses
Void ratio	Increase	Decrease	63, 122	

Asphalt Mixtures—Fundamental measures of shear strength of asphalt mixtures have been examined by numerous investigators (8, 27, 29, 39, 40, 44, 45, 46, 47, 66, 67, 68, 69, 80, 89, 112). Most of these investigators used a triaxial apparatus to evaluate the properties c and ϕ. Table 7 gives some of the more significant test results.

Indirect Strength Measures

Prior discussion has examined the strength of pavement materials from a rather fundamental point of view. Other more empirical studies and procedures have been extensively used to evaluate the relative strengths of pavement materials. These test procedures have certain advantages because of low cost, simplicity, speed, and, oftentimes, adaptability to field measurements. The tests include, among many others,

TABLE 6

SHEAR STRENGTH OF COHESIONLESS SOILS

Variable	Change in Variable	Effect on Strength	Reference	Remarks
Loading				
Type of test			63	ϕ is greater by about 2 deg from direct shear than from triaxial tests
Intermediate principal	Increase	Constant	63	
stress	Increase	Increase	26	Based primarily on theory
Rate of loading	Increase	Constant	63	
Number of load appli-	Increase	Increase	63	For loose sands only
cations	Increase	Decrease	63	For dense sands only
Confining pressure	Increase	Decrease	63, 114	
(minor principal stress)	Increase	Decrease	63, 114	
Mixture				
Void ratio	Increase	Decrease	63, 114, 122	
Aggregate gradation	Increase	Increase	63, 114	Increase means well graded
Aggregate angularity	Increase	Increase	63, 114	
Aggregate mineralogy			63	No effect unless micaceous or low crush resistance
Environmental				
Saturation			63, 114, 122	No effect

TABLE 7

SHEAR STRENGTH OF ASPHALT MIXTURES

Variable	Change in Variable	Effect on Strength	Reference	Remarks
Loading				
Rate of loading	Increase	Increase in c	8, 40, 47, 68	
	Increase	Decrease in ϕ	40, 47, 68	
	Increase	Constant ϕ	27	Theoretically constant ϕ (68)
Mixture				
Aggregate angularity	Increase	Increase in ϕ	27, 37, 40	
	Increase	Increase in c	27	
Aggregate gradation	Increase	Increase in ϕ	27	Increase means coarse to fine
Asphalt content	Increase	Optimum in c	8, 27, 40	
	Increase	Decrease in ϕ	8, 40	
Asphalt viscosity	Increase	Constant ϕ	27, 47	
	Increase	Increase in c	27, 47	
Environmental				
Temperature	Increase	Decrease in c	27, 40, 47	
	Increase	Increase in ϕ	40, 47	Slight
Immersion in water		Decrease in c	8	
		Decrease in ϕ	8	

California bearing ratio, cone penetrometer, Marshall stability, Hubbard-Field stability, and gyratory shear methods. These methods have served, and will continue to serve, in the judicious design of flexible pavements by empirical means. However, the possibility of their use in more rational design procedures is remote.

DEFORMABILITY

To enable the accurate estimation of stresses and strains in a pavement structure and the permanent cumulative deformations associated therewith, we must characterize the complex stress-strain-time behavior of pavement materials before the limiting or failure domain is reached. Of interest are two perhaps inseparable components of response: recoverable deformations and irrecoverable deformations.

Existing knowledge of the recoverable component of response is now quite well advanced and is sufficient for many useful engineering analyses of real-world problems including the dynamic response of pavements to traffic loads. At the same time, very little is known of the irrecoverable response of pavement materials to load not only for creep under long-time loading but also for the accumulation of irrecoverable deformations—both volume change (densification) and shape change—due to repetitive loading. Because rutting is one of the most prevalent forms of distress in flexible pavements and because a thorough knowledge of the irrecoverable response of pavement materials is essential to a rational analysis of the rutting problem, it would appear that the major deficiency in our current ability to characterize the mechanical behavior of these materials is in the area of irrecoverable deformations. However, recent analyses by Barksdale and Leonards (5) represent an important first step in developing an ability to predict rut depths. Operational moduli used in their linear viscoelastic analysis were derived from the results of creep tests (creep compliance as defined later) and repeated load tests (total axial strain versus number of load applications).

A second major deficiency is that most experimental investigations of mechanical response have used simplified loading patterns such as uniaxial stresses, which are quite different from the complex three-dimensional pattern to which materials are subjected in pavement structures.

Characterizing Deformability

The principal means for characterizing mechanical response is through use of a modulus defined as the ratio of stress to strain and a strain ratio, such as Poisson's ratio, for relating strains in mutually perpendicular directions. For complex materials the modulus or ratio will not be invariant but will depend on the level of stress (or strain) at which it is evaluated. Thus, additional definition is required, and the terms tangent modulus and secant modulus are often used. The tangent modulus is the slope of a tangent to the stress-strain curve; and, if the curve is not linear, the particular level of stress must be specified for the measure to be meaningful. The secant modulus is the slope of a line connecting two points of the stress-strain curve; and, to be meaningful, both points must be explicity stated. Most moduli used to characterize pavement materials are secant moduli.

Meaningful characterization of pavement materials can be accomplished in one of two ways depending on the complexity of the behavior. If the material behaves as any one of a number of idealized models, it is possible to devise suitable test procedures that evaluate fundamental material properties. These properties are sufficient in themselves to fully characterize the behavior of the material in a structure subject to any loading condition that does not induce failure. In the case of a linearly elastic isotropic material, these properties are Young's modulus, E, and Poisson's ratio, μ. Young's modulus would be evaluated by the ratio of stress to strain and would be constant for all conditions and all levels of stress, strain, and time.

On the other hand, as is the usual case for pavement materials, it may be impossible to identify an idealized model that adequately represents the measured material response. Measured moduli thus represent derived material properties for these materials. It is well to emphasize that such derived material properties are extremely useful in engineering analyses but only if the loading conditions in the structure are adequately simulated by the test procedure (or if the boundary conditions of the test closely approximate those on a small element within the structure).

In summary, then, mechanical response of pavement materials is most often specified in terms of a secant modulus that, by virtue of the complexity of these materials, is a derived property useful for analysis only when the testing procedure adequately simulates the real-world loading conditions.

Deformability Under Constant Loads—Creep tests and relaxation tests are used to examine the behavior of time-dependent materials under constant loading. In the creep test, a stress, σ_0, is applied "instantaneously" and maintained constant throughout the test duration, and the resulting strain is measured as a function of time, $\epsilon(t)$. A measure of response is the creep compliance, D(t), where

$$D(t) = \epsilon(t)/\sigma_0 \tag{8}$$

A more conventional representation of this behavior is given by the creep modulus, $E_c(t)$, where

$$E_c(t) = \sigma_0/\epsilon(t) \tag{9}$$

In the stress relaxation test, an instantaneously applied strain, ϵ_0, is maintained at a constant level, and the stress is observed as a function of time, $\sigma(t)$. The time-dependent relaxation modulus, $E_r(t)$, is calculated by

$$E_r(t) = \sigma(t)/\epsilon_0 \tag{10}$$

In general, the creep modulus and the relaxation modulus will be unequal. However, if the material response is that of a linear viscoelastic material, it is possible by means of analytical techniques to convert from one modulus to the other (30, 31, 74). Neither the creep modulus nor the relaxation modulus would seem to be in a form suitable for pavement analysis and design except possibly to predict long-term deformations under static load.

Deformability Under Uniformly Varying Loads—The uniformly varying load category includes both constant-rate-of-strain tests ($d\epsilon/dt = c$) and constant-rate-of-stress tests ($d\sigma/dt = c$). Time-dependent response to these test conditions can be given in terms of a secant modulus, $E_S(t)$, or a tangent modulus, $E_T(t)$, where

$$E_S(t) = \sigma(t)/\epsilon(t) \tag{11}$$

and

$$E_T(t) = d\,\sigma(t)/d\,\epsilon(t) \tag{12}$$

In general the moduli obtained from stress-controlled tests will not equal those determined from strain-controlled tests. If the behavior is linearly viscoelastic, however, the principle of superposition may be used to convert from one to the other or to creep or relaxation moduli. The magnitudes of these moduli will generally depend on the rate of loading.

Because neither the constant-rate-of-stress nor the constant-rate-of-strain test accurately simulates pavement loadings, direct use of these moduli for pavement analysis and design is questionable. Their use to date, as with creep and relaxation tests, has been primarily to determine the extent to which pavement materials are (a) linearly viscoelastic and (b) thermorheologically simple. These test methods have also been used to examine the viscoelastic response of asphalt paving slabs under creep loading (102) and to evaluate the constants of various mechanical models of material behavior (76).

Deformability Under Sinusoidal Loads—A particularly convenient form of laboratory loading is that in which the loads vary sinusoidally with time. Consider the case where the applied stress level σ is given by

$$\sigma(t) = \sigma_0 \sin \omega t \tag{13}$$

where σ_0 is the stress amplitude, ω is the frequency, and t is time. The strain response to such a loading pattern in the steady-state condition will likewise be sinusoidal but will lag the stress by an angle ϕ, that is,

$$\epsilon(t) = \epsilon_0 \sin (\omega t - \phi) \tag{14}$$

where $\epsilon(t)$ is the strain at time, t; ϵ_0 is the strain amplitude; and ϕ is the phase angle or lag.

Under these conditions, it is possible to relate the stresses and strains by a complex number called the complex modulus, E^*, such that

$$E^* = E' + iE'' \tag{15}$$

where E' is the real part of the complex modulus, and E'' is the imaginary part of the complex modulus, and

$$E' = \frac{\sigma_0}{\epsilon_0} \cos \phi \tag{16}$$

and

$$E'' = \frac{\sigma_0}{\epsilon_0} \sin \phi \tag{17}$$

E', the in-phase component of the modulus, represents stored recoverable energy and is called the storage modulus. E'', the in-quadrature component of the modulus, represents the energy lost by internal friction within the material and is called the loss modulus.

The absolute value of the complex modulus, $|E^*|$, is

$$|E^*| = \sigma_0/\epsilon_0 \tag{18}$$

The ratio of the stress and strain amplitudes (Eq. 18) has also been called the stiffness modulus (6). The complex modulus is fully specified in terms of its absolute value, $|E^*|$, and its argument, the phase lag, ϕ.

For a linear viscoelastic material, the complex modulus is a fundamental material property that varies with the frequency of load application (87). A knowledge of the complex modulus and the complex Poisson's ratio, both of which may vary with frequency, is sufficient to characterize the behavior of a linear, isotropic, viscoelastic material. Once these properties are established, it is theoretically possible to describe the response of the material to any given loading pattern by means of Fourier and Laplace transforms. Papazian (87) has discussed the utility of such measures of response and indicated how the complex modulus can be evaluated by means of dynamic tests or static (creep) tests. The latter requires a graphical procedure for evaluation.

Another means for determining a dynamic modulus under sinusoidal loading is to measure the resonant frequency of a vibrating specimen-machine system. The modulus so determined is the real component, E', of the complex modulus and is always less than the absolute value of the complex modulus for nonzero phase lags (92). Hardin and Drnevich (35, 36) have extensively tested soils in this way and have characterized their behavior in terms of the shear modulus and the damping ratio, D, which is directly proportional to the phase lag, ϕ.

Deformability Under Pulsating Loads—Others, most notably Seed and others (106, 107), have subjected specimens to dynamic loads of a pulsating nonreversing form. Most of these repeated load tests have been of the triaxial compression variety. The measure of response is the modulus of resilient deformation, M_r, where

$$M_r = \sigma_d/\epsilon_r \qquad (19)$$

In this equation, σ_d is the repetitively applied deviator stress and ϵ_r is the resilient or recoverable axial strain corresponding to a specific number of load applications.

A major advantage of this method of testing is that the loading pattern may be selected to simulate that occurring in the pavement structure. Hence, the modulus of resilient deformation, though not usually conceived as a fundamental material property, can be used directly in analytical investigations.

Some Concluding Remarks—This discussion indicates that one may characterize the behavior of pavement materials in numerous ways depending in part on the nature of the problem and in part on personal preferences. It must be emphasized, however, that in most cases pavement materials do not possess idealized properties and that the measured properties are often significantly influenced by the test procedures and equipment. It is important, therefore, for laboratory procedures to simulate to as great a degree as possible actual field loading conditions. Test procedures that result in nearly homogeneous stress and strain states are necessary to investigate the properties of a small volume element.

Cohesive Soils

Dehlen (16) states:

> A clay subjected to stress shows immediate and time-dependent recoverable and permanent strains, the immediate strains being predominant under short-duration loads, and the permanent strain per cycle decreasing to an insignificant amount after many cycles of stress. Stress history may have a significant effect on the response. The response is markedly non-linear.

The nonlinear response of cohesive soils is evidenced in two ways. First, the stiffness of these materials is dependent on the initial stress state and increases as the effective mean principal stress increases. Second, and more important, the stiffness decreases with an increase in the incremental stress amplitude (deviator stress in triaxial tests). These and other effects of testing variables on the stiffness and damping of cohesive soils are given in Tables 8 and 9. The results of most investigations of the major effects are in common accord.

TABLE 8

STIFFNESS OF COHESIVE SOILS

Variable	Change in Variable	Effect on Stiffness	Reference	Remarks
Loading				
Number of cycles	Increase	Decrease	35, 117	
	Increase	Minimum	79, 106	Minimum at 1 to 5,000 cycles
Incremental strain amplitude	Increase	Decrease	35, 50, 117	Rate of decrease depends on maximum stiffness and shear strength
Incremental stress amplitude	Increase	Decrease	16, 17, 53, 63, 79, 106	Rapid decrease at low stresses
Effective initial mean principal stress	Increase	Increase	35, 50, 53, 63	Effect depends on stress or strain amplitude
Transverse stress			16, 17	No effect
Initial octahedral shear stress			35	Effect negligible after 10 cycles
Frequency of loading	Increase	Increase	10	Effect minor above 0.1 cps
	Increase	Increase	35	
Strain rate	Increase	Increase	63	
Overconsolidation ratio	Increase	Increase	35, 63	
			50	Any effect can be explained on basis of effective pressure and void ratio
Stress path			63	Large dependency
Mixture				
Soil disturbance	Increase	Decrease	42, 63	
Void ratio	Increase	Decrease	35	
	Increase	Decrease	50	Maximum effect at low confining pressure
Dispersion	Increase	Decrease	63, 79	At small strains
Structure			35	Little effect on maximum shear modulus
Degree of saturation at compaction	Increase	Decrease	79	Modulus of resilient deformation
	Increase	Maximum	85	
Plasticity	Increase	Decrease	63	
Compaction energy	Increase	Maximum	85	Impact compaction
	Increase	Increase	42	
Environmental				
Aging	Increase	Increase	63	
Degree of saturation	Increase	Decrease	10, 35, 42, 79	
Time (thixotropy)	Increase	Increase	35, 79	Recovery after high amplitude cyclic loading or many load cycles
Densification	Increase	Increase	79	
Time (during secondary compression)	Increase	Increase	50	Bentonite

TABLE 9

DAMPING OR PHASE ANGLE OF COHESIVE SOILS

Variable	Change in Variable	Effect on Damping or Phase Angle	Reference	Remarks
Loading				
Number of cycles	Increase	Decrease	35	Up to about 50,000 cycles beyond which damping increases
Incremental strain amplitude	Increase	Increase	35	Very rapid increase but tends to reach a maximum at large strains
Effective initial mean principal stress	Increase	Decrease	35	
Initial octahedral shear stress	Increase	Increase	35	
Frequency of loading	Increase	Increase	35	Effect minor above 0.1 cps
Mixture				
Void ratio	Increase	Decrease	35	
Molding water content	Increase	Minimum	85	
Compaction energy	Increase	Minimum	85	Impact compaction
Environmental				
Time (thixotropy)	Increase	Change	35	Recovery after high amplitude cyclic loading

The following observations are also of importance in understanding the behavior of cohesive soils:

1. The cohesive soil investigated by Dehlen (17) was found to be initially cross-isotropic with the horizontal stiffness exceeding the vertical stiffness. In addition, a significant degree of stress-induced cross isotropy was observed.
2. Depending on the testing conditions, Poisson's ratio remained constant or increased slightly with increasing applied compressive stresses but was independent of the transverse stress (17).
3. The effect of the void ratio, e, on the maximum (low strain) dynamic shear modulus for void ratios less than about two is given by (35)

$$F(e) = (2.973 - e)^2/(1 + e) \tag{20}$$

4. Soil structure as affected by molding water content and type of compaction can have a significant effect on stiffness, as discussed by Monismith et al. (79):

> . . . samples compacted "wet of optimum" for a particular compactive effort by static means have similar resilience characteristics to those compacted "dry of optimum" by kneading compaction and subsequently soaked to a similar degree of saturation. . . [and hence have] resilient characteristics similar to those observed in field specimens for the same conditions of test.

5. As given in Table 8, most investigators have found the behavior of cohesive soils to be highly nonlinear. At the same time, Pagen and Jagannath (85) observed linear viscoelastic behavior in unconfined axial creep for unsaturated compacted clays up to an axial stress of 20 to 24 psi. Application of confining pressures was found to extend the range of linearity.
6. Coffman (10) found that the complex moduli obtained from dynamic tests tend to be larger than those obtained from creep tests. Although this difference was attributed to the effects of stress history, it may have been related to a disregard of inertial effects associated with the dynamic tests.

Cohesionless Soils

Regarding cohesionless soils, Dehlen (16) states:

> . . . the response of sand is primarily instantaneous and time independent. Large permanent strains may occur during the first cycle of stress, but the behavior becomes almost elastic after many cycles. The effects of stress history are generally less marked than in the case of clays, and the stress-strain response is non-linear.

Tables 10 and 11 give the effects of some testing variables on the stiffness and damping respectively of cohesionless soils. A primary loading variable is that of the initial mean effective principal stress, $\bar{\sigma}_0$, which affects the stiffness, S, as follows:

$$S = K\ \sigma_0^n \tag{21}$$

where K is a constant and n is an exponent that varies from about 0.4 to 1.0 (63). A larger exponent is observed for less dense materials (63) and for larger strain amplitudes (35). Increases in the incremental stress amplitude cause reductions in stiffness, another evidence of the nonlinearity of these materials. Effects of the deviatoric component of the initial stress state are negligible for repeated loading.

The stress-strain curve under cyclic loading is characterized by a hysteresis loop that stabilizes after 10 to 50 cycles following which there is little or no additional permanent strain for each cycle of loading (63). During the initial cycles, however, a net compressive strain is developed under triaxial compression loading (63). The hysteresis loop is an indication of the degree of damping in the material.

When sheared, a loose sand generally contracts in volume until failure is approached, at which time expansion is observed. Dense sands expand even at low strains and continue to expand as failure is approached. For sands of all initial densities, the rate of expansion increases near failure (58, 63).

Cohesionless soils are probably more isotropic than other pavement materials as evidenced in tests on Ottawa sand by Ko and Scott (59). Ko and Scott also found that

TABLE 10

STIFFNESS OF COHESIONLESS SOILS

Variable	Change in Variable	Effect on Stiffness	Reference	Remarks
Loading				
Incremental stress amplitude	Increase	Decrease	63	
Incremental strain amplitude	Increase	Decrease	35	Rapid decrease
Number of cycles	Increase	Increase	35, 63	Approaches a maximum
Load duration	Increase	Decrease	103	Pulsating loads
Loading rate or frequency	Increase	Constant	63	No effect after first few load cycles
	Increase	Constant	35, 63	0 to a few hundred cps
Initial effective mean principal stress	Increase	Increase	35, 63	
Initial octahedral shear stress	Increase	Decrease	35	Very small effect after 10 load cycles
Mixture				
Void ratio	Increase	Decrease	35, 57, 63, 121	
Environmental				
Degree of saturation	Increase	Constant	35, 63	Effective stresses must be used

TABLE 11

DAMPING OR PHASE ANGLE OF COHESIONLESS SOILS

Variable	Change in Variable	Effect on Damping or Phase Angle	Reference	Remarks
Loading				
Incremental strain amplitude	Increase	Increase	35	Very rapid increase but tends to reach a maximum at large strains
Number of cycles	Increase	Decrease	35	Up to about 50,000 cycles beyond which damping increases
Initial effective mean principal stress	Increase	Decrease	35	
Initial octahedral shear stress	Increase	Increase	35	
Frequency of loading	Increase	Constant	35	0 to a few hundred cps
Mixture				
Void ratio	Increase	Decrease	35	
Environmental				
Degree of saturation	Increase	Constant	35	

behavior in their soil test box was quite different from that observed in conventional triaxial tests (59) and in one-dimensional compression tests (57).

Finally, the influence of void ratio for ratios less than about two was found by Hardin and Drnevich (35) to be adequately described by Eq. 20.

Untreated Granular Aggregate

The behavior of untreated granular aggregate is quite similar to that of the smaller sized cohesionless soils. However, a comparison of Table 12 for untreated granular aggregates with Table 10 for cohesionless soils reveals some behavioral differences such as observed for the effects of incremental stress level. Whether these differences indicate fundamental differences in behavior of the two types of materials or just differences in test equipment and procedures is unknown.

In any case the effect of initial confining pressure or mean initial effective principal stress is of paramount importance. The modulus of resilient deformation, M_r, is related to the initial stress state as follows:

$$M_r = K \sigma_3^n \tag{22}$$

and

$$M_r = K' \bar{\sigma}_0^{n'} \tag{23}$$

where K, n, K', and n' are material constants, σ_3 is the confining pressure in a triaxial test, and $\bar{\sigma}_0$ is the mean initial effective principal stress (17, 48, 53, 79). The constant, n, is said to vary from 0.35 to 0.55 and n' from 0.35 to 0.6 (17). Any nonlinearities under cremental loading would appear to be relatively insignificant as long as the stress increments are small and shear failure is not approached.

Poisson's ratio, μ, increases with (a) decreasing confining pressure, (b) increasing incremental stresses, (c) decreasing fines, and (d) decreasing degree of saturation (48). It is apparently affected little by density variations and can be estimated from an equation of the following type (48):

$$\mu = A_0 + A_1(\sigma_1/\sigma_3) + A_2(\sigma_1/\sigma_3)^2 + A_3(\sigma_1/\sigma_3)^3 \tag{24}$$

in which A_i is constant.

Asphalt Paving Mixtures

About asphalt paving mixtures, Dehlen (16) states the following:

> The response of asphalt concrete to stress is influenced to a pronounced degree by time. Asphalt concrete under stress exhibits immediate followed by time-dependent strain, both of which may be partly recoverable and partly permanent. The time-dependent response may be viscous or non-viscous. Under stresses of short duration, such as experienced under moving traffic, the in-

TABLE 12

STIFFNESS OF UNTREATED GRANULAR AGGREGATE

Variable	Change in Variable	Effect on Stiffness	Reference	Remarks
Loading				
Initial confining pressure	Increase	Increase	17, 48, 79	Triaxial compression
Initial effective mean principal stress	Increase	Increase	17, 79	
Incremental stress level	Increase	Constant	17, 79	As long as shear failure does not occur
	Increase	Increase	48	Slight increases as long as shear failure does not occur
Loading rate or frequency	Increase	Increase	10, 12, 79	Small increase
Load duration	Increase	Constant	48	0.1 to 0.25 sec
Number of cycles	Increase	Constant	48	After 50 to 100 applications of in-service stress levels
Drainage	Increase	Constant	48	
Mixture				
Void ratio	Increase	Decrease	12, 48	
	Increase	Decrease	10	At low moisture contents
	Increase	Increase	10	At high moisture contents
Angularity and surface roughness	Increase	Increase	48	
Fines	Increase	Decrease	48	
			38, 79	Minor effect
Compaction water content	Increase	Decrease	53	
Environmental				
Degree of saturation	Increase	Decrease	10, 12, 38, 48, 79	

stantaneous strains form a large proportion of the total. In a material exhibiting these time-dependent and permanent strains, stress history will influence the response. For adequately designed pavements, previously subjected to many cycles of stress, the permanent strains due to a single stress cycle are very small. The moduli of asphalt concrete vary with stress intensity, and the moduli in tension differ from those in compression. The material is thus non-linear, and this is true even at low strains. The non-linearity becomes more pronounced with increasing temperature.

These introductory remarks certainly indicate the complexity of the mechanical behavior of asphalt paving mixtures. Tables 13 and 14 give the effects of some of the variables that influence this behavior. Of particular interest and perhaps some controversy is the extent of linearity. The effects of stress amplitude as given in Table 13 are certainly inconclusive and indicate observations of both linear and nonlinear behavior.

Monismith and others (74) have observed reasonably linear behavior for uniaxial loading as long as the strains are less than 0.1 percent. Sayegh (98) observed that the domain of linearity for a particular mix was limited to deformations of less than 4×10^{-5},

TABLE 13

STIFFNESS OF ASPHALT PAVING MIXTURES

Variable	Change in Variable	Effect on Stiffness	Reference	Remarks
Loading				
Frequency of loading	Increase	Increase	6, 10, 53, 87, 90, 93, 98	Sinusoidal loading, stiffness approaches asymptote at high frequencies
	Increase	Decrease	14	Pulsating (incomplete stress relaxation)
Incremental stress amplitude	Increase	Constant	79	100 to 125 psi flexure
	Increase	Constant	53, 111	17.5 to 70 psi compression
	Increase	Constant	87	4 to 33 psi compression
	Increase	Constant	49	20 to 35 psi compression
	Increase	Decrease	53	50 to 200 psi flexure (80 F)
	Increase	Decrease	92	Rate of decrease less with increasing amplitude
	Increase	Decrease	32, 115	
Number of cycles	Increase	Decrease	53	
	Increase	Constant	33	Some effect at higher temperature (+30 C)
Strain history			92	No effect on dynamic torsional stiffness
Initial confining pressure	Increase	Increase	32, 115	
Mixture				
Air void content	Increase	Decrease	14, 19, 49, 52, 111	
	Increase	Decrease	6	Approximately linear relation between log stiffness modulus and void content
Asphalt content	Increase	Optimum	49	
	Increase	Decrease	111	4 to 6 percent asphalt at constant air voids
	Increase	Variable	111	For constant compaction, effect is variable depending on air voids and temperature
	Increase	Decrease	93	
Asphalt viscosity	Increase	Increase	93, 111	
Filler content	Increase	Increase	93, 111	
Environmental				
Temperature	Increase	Decrease	6, 10, 79, 92, 93, 98	Stiffness approaches asymptote at low temperatures

TABLE 14

DAMPING OR PHASE ANGLE OF ASPHALT PAVING MIXTURES

Variable	Change in Variable	Effect on Damping or Phase Angle	Reference	Remarks
Loading				
Frequency of loading	Increase	Maximum	87	Maximum at 4 rad/sec in range of 0.1 to 100 rad/sec
	Increase	Increase	53	4 to 14 cps
	Increase	Decrease	6	
			10	Effect variable depending on temperature
Mixture Variable				
Air voids	Increase	Decrease	6	Opposite effect observed if mix is initially weak
Environmental				
Temperature	Increase	Increase	53	Range of 40 to 70 F
	Increase	Increase	6	Maximum observed at high temperatures
			10	Variable depending on frequency

an extremely small range indeed. On the other hand, Krokosky, Tons, and Andrews (61) observed nonlinear viscoelastic behavior. Coffman, Ilves, and Edwards (11) observed that, for practical purposes, the moduli in tension and compression are equal, but others (14, 30) have found that stiffness in compression exceeds that in tension. Nonlinear behavior of the stiffening type has been observed under low levels of stress (49), short loading times (49), and compressive stresses (17). A softening nonlinearity has been observed under high levels of stress (49), long loading times (49), and tensile stresses (17). Pell and Taylor (93) aptly summarize the situation as follows:

> The non-linear behavior of a mix would appear to depend upon the properties of that mix and the environmental and loading conditions to which it is subjected, so making it difficult to give a particular value of stress or strain above which all mixes will exhibit non-linear behavior.

Table 15 gives some of the factors that affect the range of linearity of asphalt paving mixtures.

Asphalt mixtures in situ would appear to be anisotropic because of the layering and particle orientation inherent in the construction process. Dehlen and Monismith (17) concluded that the mixture they investigated was initially cross-isotropic with the horizontal stiffness exceeding the vertical stiffness. A significant degree of stress-induced cross isotropy was also observed. On the other hand, Coffman and others (11) concluded that, for practical purposes, asphalt concrete is isotropic in compression at the phenomenological level.

TABLE 15

RANGE OF LINEARITY OF ASPHALT PAVING MIXTURES

Variable	Change in Variable	Effect on Range of Linearity	Reference	Remarks
Loading				
Frequency of loading	Increase	Increase	93	
Type of loading			11	Linearity greater in compression than flexure
Mixture				
Void content	Increase	Decrease	93	
Asphalt content	Increase	Increase	93	
Asphalt viscosity	Increase	Increase	93	
Filler	Increase	Increase	93	
Environmental				
Temperature	Increase	Decrease	16, 17, 93	

Kallas and Riley (53) have demonstrated that observed moduli are sensitive to the testing procedure. They state:

> The difference between moduli determined by repeated load flexure and dynamic complex modulus test procedures may in part be attributed to different deformational responses: essentially recoverable for the complex modulus test procedures but only partially recoverable for the repeated load flexure test procedures.

Limited information is available concerning the effects of the testing variables on Poisson's ratio:

1. Poisson's ratio decreases for increasing sinusoidal frequencies above 4 cps (53, 98);
2. Poisson's ratio remains constant or increases slightly with increasing applied compressive stress but is independent of transverse stress (17);
3. Poisson's ratio is relatively insensitive to mixture type (53);
4. Poisson's ratio decreases as temperature decreases below 70 F (53, 98); and
5. Poisson's ratio is a real number that increases from 0.1 for high frequencies, low temperatures, and small deformations to 0.5 for low frequencies, high temperatures, and large deformations (98).

For a given bituminous mixture and variable temperature and frequency, there is a unique relationship between stiffness and phase angle such that there is only one phase angle corresponding to a given stiffness. This has been validated by complex modulus evaluations (6, 98).

Finally, it appears that asphalt paving mixtures are, to an engineering approximation, thermorheologically simple (6, 13, 60, 61, 78, 98). This means that there is an equivalence between time of loading and temperature, which enables the stiffness response of such a material to be presented in terms of a master curve (stiffness versus reduced time) and a shift factor curve (shift factor versus temperature). Such a property not only enables a simplified presentation of stiffness response but also enables an estimation of stiffness properties for an extended range of temperatures and times of loading beyond which it is inconvenient to perform laboratory tests.

Estimating Stiffness

Cohesive and Cohesionless Soils—By means of the equations and graphs presented by Hardin and Drnevich (36), it is possible to estimate for design purposes the dynamic shear modulus and the damping of both cohesive and cohesionless soils. Factors such as initial void ratio, overconsolidation ratio, mean effective principal stress, effective vertical stress, frequency of loading, number of loading cycles, plasticity index, and static strength parameters in terms of effective stresses ($\bar{c}$ and $\bar{\phi}$) are treated as independent variables in the estimation process.

A somewhat less sophisticated procedure for relating the dynamic stiffness, E, to an empirical strength measure, CBR, is

$$E = 100\ \text{CBR} \tag{25}$$

where E is the dynamic stiffness in kg/cm^2. Equation 25 represents a rough correlation based on field measurements and is claimed to be accurate within a factor of about two for fine-grained soils (42).

Untreated Granular Aggregate—The stiffness of an untreated granular aggregate in a pavement structure can be estimated rather crudely from a knowledge of the stiffness of supporting layers. Based on field measurements, Heukelom and Klomp (42) observed that the dynamic modulus of these materials can increase by a factor of roughly two from one compacted layer to the next. Otherwise, one could estimate the stiffness of untreated aggregate from a relationship such as that given by Eq. 23. Unfortunately, however, there is limited general knowledge concerning the relationships of the constants of Eq. 23, K' and n', to the mixture properties, and the work of individual investigators would have to be reviewed to obtain suitable estimates of these constants.

Asphalt Paving Mixtures—Bazin and Saunier (6) have observed that:

> Within the range of linear behavior in bending, for mixes with correct binder contents and normal void contents (4-8%) all mix variables tested had only a small effect on the complex modulus as compared to the effects of binder type, temperature, and rate of loading.

Observations such as this have led investigators to search for procedures to enable prediction of the dynamic modulus of asphalt paving mixtures based on routine test properties.

The work of Van der Poel (125, 126) is certainly notable in this respect. Based on extensive static and dynamic testing, Van der Poel developed a nomograph useful for predicting the stiffness of pure bitumens, S_{bit}, such that

$$S_{bit} = f\,(\text{frequency of loading, temperature, penetration of extracted bitumen, and ring-and-ball softening point of extracted bitumen}) \quad (26)$$

where f is some function. A second nomograph enabled the estimation of mixture stiffness, S_{mix}, from S_{bit} and the volume concentration of mineral aggregate, C_v.

Heukelom and Klomp (43) slightly modified Van der Poel's nomograph for obtaining S_{bit} and suggested that S_{mix} could be obtained as follows:

$$S_{mix}/S_{bit} = \{1 + 2.5\,C_v/[n(1 - C_v)]\}^n \quad (27)$$

where

S_{mit} = mixture stiffness in kg/cm^2,
S_{bit} = bitumen stiffness in kg/cm^2 obtained from nomograph at desired temperature and time of loading,
C_v = volume concentration of aggregate in the mixture (ratio of volume of compacted aggregate to volume of aggregate and bitumen), and

$$n = 0.83 \log_{10} (400{,}000/S_{bit}) \quad (28)$$

Heukelom and Klomp's method was limited to mixtures having air void contents on the order of 3 percent and C_v between 0.7 and 0.9. Van Draat and Sommer (127) have suggested that air void contents of greater magnitude can be appropriately considered by using a corrected volume concentration of aggregate, C_v', such that

$$C_v' = C_v/(1 + H) \quad (29)$$

where H is the difference between the actual air void content and 3 percent, expressed as a decimal.

Bazin and Saunier (6) have presented a nomograph similar to Van der Poel's that is valid for linear deformations in bending and that uses binder properties determined before mixing. Independent variables that are recognized include time of loading, temperature, bitumen type, and mixture void content.

Others have used standard linear regression techniques to relate the absolute value of the complex modulus (111) and the modulus of resilient deformation (25) to various properties of the mixture and its constituent materials.

Additional Investigations

The discussion has concentrated on more conventional approaches to the testing and characterization of pavement materials. It is well to point out two rather recent investigations that deviate somewhat from the conventional format and offer means for possibly more fundamental studies of mechanical behavior.

The first of these is the investigations of Ko and Scott (57, 58, 59). These investigators have developed a soil test box capable of testing a cubical sample by applying three different normal pressures to its sides. The principal stresses can be varied independently or by means of a stress control device that is a mechanical-hydraulic analog of an octahedral plane in the principal stress space. With this equipment, it is possible to vary only the octahedral shear stress while maintaining a constant hydrostatic stress to study the true response to shearing stresses. The three principal strains are measured with this device, and an independent measurement is made of the volume change of the soil sample. Reputed advantages of this device include the following: (a) a homogeneous stress state is produced, (b) the nature of the stress path that can be developed is unlimited, (c) the device is stress-controlled rather than strain-controlled, and (d) it is applicable for both loading and unloading tests. Although the adoption of this equipment for routine testing is difficult to envision, it does offer a means for research investigations into the three-dimensional response of pavement materials.

The second investigation is that of Dehlen and Monismith (16, 17). These investigators used a triaxial testing procedure in which repeated axial stress and repeated radial stress could be independently varied and superposed over varying and unequal constant stresses in the axial and radial directions. The results of their investigations were expressed as incremental stress-strain coefficient matrices at varying reference stress states.

For axisymmetric incremental stresses the coefficient matrix relates the incremental strains, ϵ' and γ', and stresses, σ' and τ', as follows:

$$\begin{Bmatrix} \epsilon'_{rr} \\ \epsilon'_{\theta\theta} \\ \epsilon'_{zz} \\ \gamma'_{rz} \end{Bmatrix} = \begin{bmatrix} B_{11} & B_{12} & B_{13} & 0 \\ B_{12} & B_{11} & B_{13} & 0 \\ B_{31} & B_{31} & B_{33} & 0 \\ 0 & 0 & 0 & B_{44} \end{bmatrix} \begin{Bmatrix} \sigma'_{rr} \\ \sigma'_{\theta\theta} \\ \sigma'_{zz} \\ \tau_{rz} \end{Bmatrix} \tag{30}$$

This incremental formulation of the constitutive equation of a nonlinear elastic material is valid for (a) small stress and strain increments, (b) an initially cross-isotropic material with an axis of symmetry coinciding with the vertical or Z-coordinate axis, (c) a cross-isotropic stress-induced anisotropy, and (d) no coupling between shear and volumetric stresses and strains. For triaxial testing procedures Eq. 30 reduces to

$$\begin{Bmatrix} \epsilon'_{r} \\ \epsilon'_{z} \end{Bmatrix} = \begin{bmatrix} B_{11} + B_{12} & B_{13} \\ 2B_{31} & B_{33} \end{bmatrix} \begin{Bmatrix} \sigma'_{r} \\ \sigma'_{z} \end{Bmatrix} \tag{31}$$

The coefficient matrix B depends on the reference stress state at which it is determined. It was found to be approximately symmetric at hydrostatic reference stress states but significantly nonsymmetric otherwise.

The main limitation of this means for characterizing the response of nonlinear elastic materials would appear to be the limited range of stress states possible with the triaxial test procedure. That is, the triaxial test adequately defines the constitutive relations only for materials located beneath an axis of symmetry because it permits only two normal stresses to be varied independently. Complete characterization of materials outside an axis of symmetry requires investigation under three normal stresses and one shearing stress.

In addition to the investigations described here, a great deal of work has been reported involving static testing of pavement materials. Characteristic of this work are investigations by Saada (94) on compacted clays and Monismith et al. (74) on asphalt concrete. The intent of most of this testing has been to evaluate the rheological behavior of these materials under simple forms of stress. Most of the tests have been relaxation or creep tests, though some constant-rate-of-deformation or constant-rate-of-load tests have been performed. The results of these tests have been analyzed largely with the intent to (a) establish the extent of linearity, (b) examine the extent to which the material is thermorheologically simple, (c) determine values of rheological constants such as relaxation moduli and creep compliances, (d) ascertain the complexity of mechanical models necessary to accurately describe the observed behavior and assess the values of the associated constants, and (e) ascertain the applicability of the superposition principle.

Finally, it must be observed that, although this review has been limited almost solely to laboratory investigations, rather extensive field investigations have also been conducted including test pits, test tracks, and pavements in service. These field investigations serve to complement, verify, and extend the results of the laboratory investigations.

CONCLUDING REMARKS

Practitioners in the field of pavement materials characterization would probably be quick to agree that the most pressing need today is the development of a universally accepted standard means for characterizing and testing these materials. At the same time, it must be realized that the factors that have retarded the development of such standards in the past are most likely to continue to be operative in the near future. With this deficiency in mind, then, I consider the following list to represent some of the more specific, current deficiencies in our ability to adequately characterize pavement materials:

1. Understanding of what material properties are of fundamental importance to the performance of asphalt concrete pavements and what properties must be known or estimated before a rational analysis of this performance is feasible, considering the planned use of the materials and the available methods of analysis;
2. Commonly accepted and explicitly recognized criteria that would allow assessments of the utility of various test equipment and various means for characterizing behavior as a part of a rational pavement analysis procedure;
3. Knowledge of the irrecoverable or permanent component of deformation and of possible means for characterizing it for all pavement materials;
4. Understanding of the three-dimensional response of all pavement materials under realistic in-service stress states and of the degree to which behavior under simplified and commonly used stress states is similar to that under more realistic conditions (this is as true of strength measures, particularly tensile and fatigue strength of asphalt paving mixtures, as it is of deformability measures);
5. Knowledge of the possible development of slip surfaces in shear due to load repetitions of a stress level less than the shear strength (particularly for unbound granular aggregate and asphalt paving mixtures);
6. Firm basis for understanding the possibly important effects of mode of loading on fatigue behavior of asphalt paving mixtures, and ability to vary mode of loading in the laboratory in order to simulate realistic in-service loading;
7. Adequately verified comprehensive equation that allows estimates of fatigue life for asphalt paving mixtures without extensive laboratory testing;
8. Knowledge of the strength and deformation behavior of treated materials;
9. Ability to characterize in a fundamental way the volume changes in all pavement materials induced by environmental factors superimposed over stress states representative of in-service conditions;
10. Knowledge of the fatigue behavior of asphalt paving mixtures when load repetitions are combined with cyclic environmental changes (predominantly temperature and specimen support); and
11. Ability to characterize untreated granular aggregate.

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IN SITU MATERIALS VARIABILITY

George B. Sherman

The problem of material variability has plagued highway engineers since their first attempts to design a pavement structure. The heterogeneous composition of the materials required for the support of traffic loads has made it extremely difficult to develop rational theoretical design values. As a result, empirical formulas and tests have been devised and used in order that roads might be built with some semblance of order.

Since 1964 when the U. S. Bureau of Public Roads first emphasized the need for better definition of material characteristics, many states have conducted studies to evaluate variability. In situ measurements have been made and compared with design specifications. It has been recognized that highway materials are not "unique" and that they do follow statistical laws. The variabilities of materials, sampling, and testing are being isolated and analyzed. Initial steps, at least, are being taken by some states, other governmental agencies, and private consultants to make allowances for such variabilities. Specifications are being examined and in some cases changed because they do not fit the variability of the materials.

In this paper, only a limited amount of available information can be covered. The examples chosen were selected because they illustrate a problem and not because they were the best of such examples. They are intended to emphasize that designers do not deal with a uniform material.

It would seem that the designer should consider two major types of variabilities that can affect the performance of a pavement structure:

1. Variations between assumed design criteria and actual conditions during construction or during the life of the pavement; and
2. Normal variations in the materials used to construct the pavement structure.

The designer's task is to develop, by the most economical means, a highway structure that will survive in its environment to safely carry a stipulated amount of traffic. To accomplish his task he must have at present some method of estimating foundation strength; a knowledge of availability, strength, and durability characteristics of materials for constructing the structural section; and a knowledge of the performance of roads under similar environmental conditions. Under most design systems currently in use, each of his decisions is based on empirical data, semidocumented performance data, or personal experience. Part of the gap between the designer's assumptions and the final product will be discussed in this paper.

In the design of a pavement structure, use is generally made of a so-called soil profile. This consists of drilling holes and testing samples of removed material along the proposed highway alignment. From these tests and the position of the soil strata they represent, an estimate is made of the expected support value for the foundation soil that will be in place when the contractor completes his grading operation. Sometimes such estimates give a misleading indication of resulting support; but it is also surprising, when a close examination is made, how many foundation soils are actually within reasonable range of design values. Many illustrations could probably be developed along this line by using either the CBR test, the Texas triaxial test, or possibly the resilient modulus, but it will suffice to illustrate the comparison by using the California resistance (R) value.

It was estimated from soil survey data that basement soil along a 2-mile project that passes through low, rolling hills in San Benito County, California, would have a resistance value of 40 minimum. This was based on an average value of 55 and the assumption that, because of the low rolling terrain, there was only a small probability that all of the poorest materials would end up in a noncritical part of the fill. After the subgrade was completed, a series of tests showed the following:

Sample Source	R-value	Avg
Drilled holes in excavation areas	13, 47, 58, 65, 61, 68, 22, 62, 60, 36 67, 65, 47, 70, 48, 68, 68, 65	55
Top of foundation soil	56, 68, 69, 66, 65, 68, 68, 69, 75, 72 69, 61, 74, 73, 60, 35	65

The average resistance value of 65 is 25 points above the minimum design value but only 10 points above the average. However, if it is assumed that these data represent one population, the standard deviation, which is 9.4, would indicate that the variation in this material will be such that 95 percent of the material will be above 46 R-value or that roughly 96 percent will be above 40.

This example raises questions that have bothered some designers: Should all of the tests be above the design minimum? How much chance is there that additional testing will indicate more areas below minimum? What risk is there in accepting a few low values? Much of the designer's indecision is based on a lack of documented performance information as well as on a lack of statistical information to calculate the risks in accepting a few small areas of weak material. New methods of interpretation of in situ values of foundation materials are needed. Decisions to design or modify structural sections should not be made based on selective, individual tests but rather on as complete a statistical picture as is economical to obtain.

Now let us turn to the in situ variability of the resistance of materials to resilient deformation as measured by the Benkelman beam or other deflection devices. Again there are many examples in the literature that cannot be covered here. Figure 1 shows the effect of base layers on deflection values. On this project Benkleman beam deflections were obtained on the basement soil, at the top of the base, and on the top of the asphalt concrete. Attempts were made to obtain deflections on the subbase, but because of its sandy nature this was not practical or possible. However, the data do illustrate the variability of support that can be expected in the basement soil. Other such measurements have been made on other projects with sometimes a greater and sometimes a lesser degree of variability. The addition of a base material generally tends to lower the deflections but, as in this case, such a generalization is not always true. In any event, the asphalt concrete layer effect on deflection is quite notable and quite common in our measurements. This layer usually causes substantial reduction in deflection as

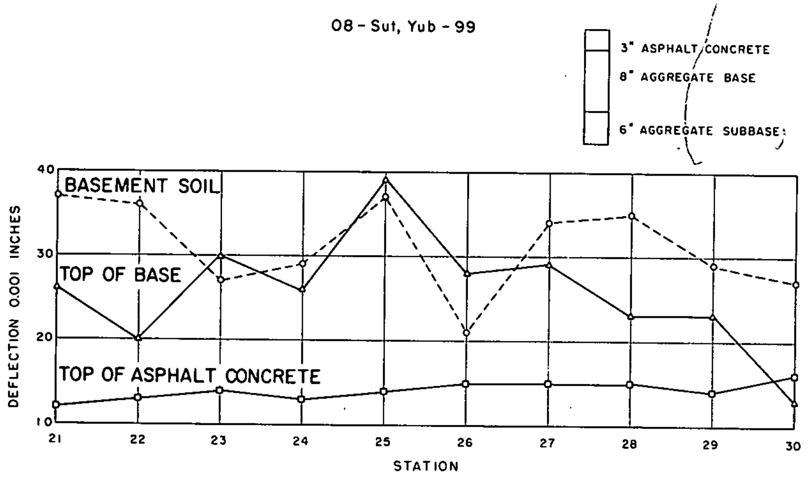

Figure 1. Effect of layers on basement soil layer.

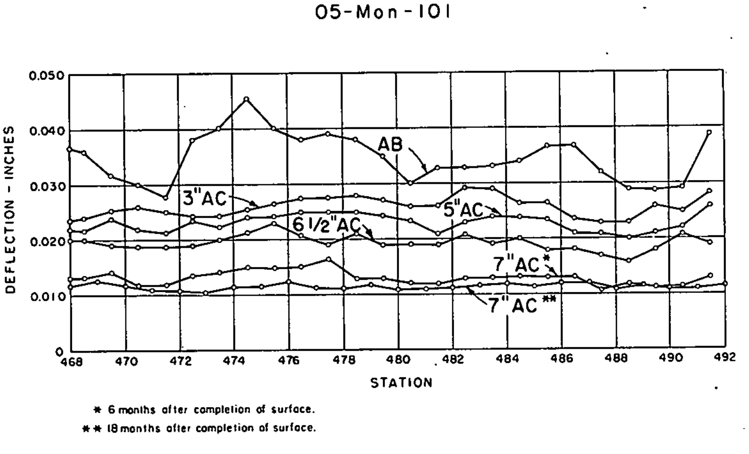

Figure 2. Effect of successive layers of asphalt concrete.

well as in the range of values. The effect of successive layers of asphalt concrete in achieving this uniform condition is shown in Figure 2.

Deflection measurements are usually fairly uniform when thicker layers of uncracked asphalt concrete are tested. As shown in Figure 3, this is not always true of older roads with thin asphalt-treated cover. In this situation the weakness of the 10-year-old double seal coat cover layer would appear to allow deflections to approach the variability of the supporting soil. A cushion course overlay consisting of 6 in. of aggregate base and 6½ in. of asphalt concrete placed on this highly resilient pavement in 1960 restored this road to a uniform tolerable deflection level and allowed for substantial increase in traffic. Deflections made in 1967 do not indicate any great change.

Although the designer assumes a certain uniformity of quality in the basement soil, he also assumes a uniformity of compaction that may or may not be present. Figure 4

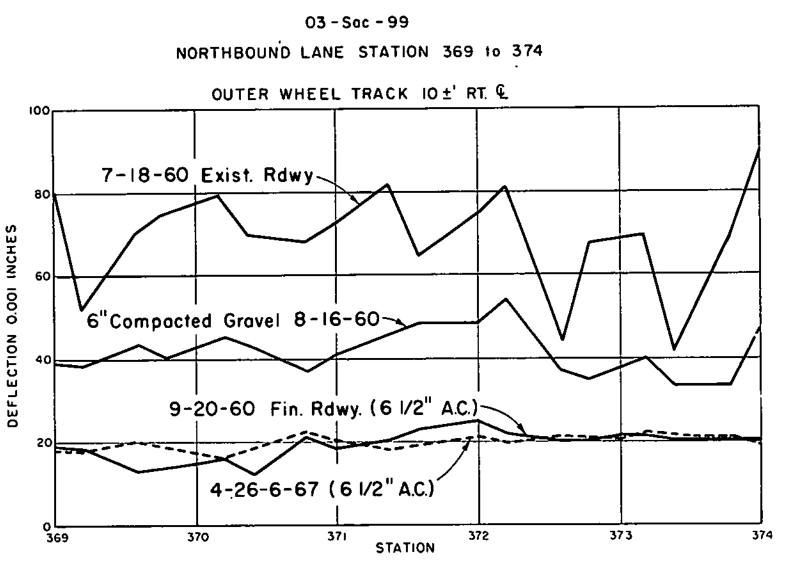

Figure 3. Effect of age on deflections.

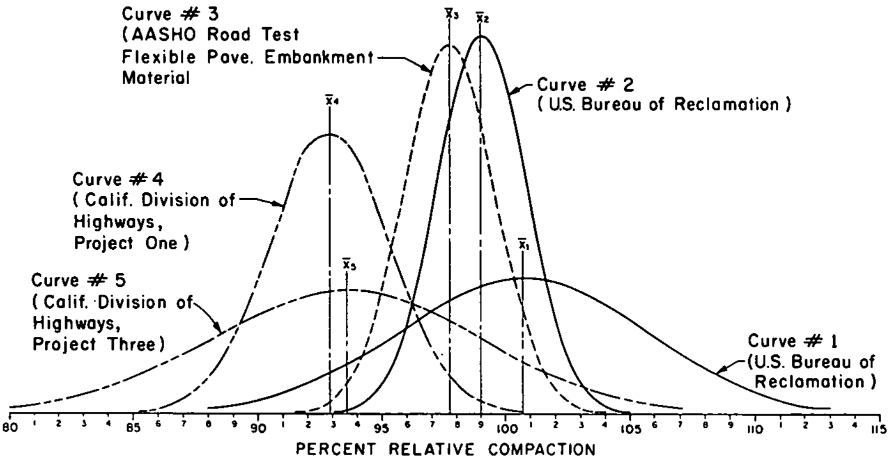

Figure 4. Comparison of uniformity of compaction.

shows the variability that has been encountered when compaction tests were made after the completion of a grade. As might be expected, the more uniform the material is, the narrower the range of values that are encountered on a particular project is. The more heterogeneous the material is, the greater the standard deviation and the spread of test results are. It is also noteworthy that the standard deviation bears no real relationship to the average value. The curves shown in the figure were obtained from data published by the U. S. Bureau of Reclamation, the AASHO Road Test, and the California Division of Highways (1). Resemblance of curves between two agencies such as the California Division of Highways and the Bureau of Reclamation indicates that these data are not uncommon and probably span the compaction variation of many subgrades. The specifications and compaction test methods used to obtain the data for these curves are given in Table 1.

There are many other variations between assumed design criteria and in situ conditions. Some of these are the variations between the material that the designer thought might be used and the material that the contractor actually used. This is particularly true of pit run material such as imported borrow or subbase. There are generally variations between the assumed moisture and density conditions that might develop in the road and those that actually occur. Such variations are affected not only by assumed and actual environmental conditions but also by improper compaction, poor drainage, or materials of inferior durability. The determination of moisture and density of test

TABLE 1

DATA FOR COMPACTION CURVES SHOWN IN FIGURE 4

Agency	Material	Compaction Test Method	Average Compaction	Standard Deviation	Approximate Percent Less Than Minimum Specified Limit
U. S. Bureau of Reclamation	Heterogeneous	Proctor E-11	100.7	5.0	29.5[a]
U. S. Bureau of Reclamation	Uniform	Proctor E-11	99.0	1.8	28.9[a]
AASHO Road Test	Flexible pavement embankment	AASHO T-99	97.7	1.9	7.8[b]
California Division of Highways	Uniform	Calif. 216	92.9	2.4	11.3[c]
California Division of Highways	Heterogeneous	Calif. 216	93.6	5.5	25.6[c]

[a]98 percent minimum relative compaction limit.
[b]95 percent minimum relative compaction limit.
[c]90 percent minimum relative compaction limit.

specimens to obtain support values for design is one of the difficult areas yet to be investigated to a positive conclusion. Currently there is a strong leaning toward the use of soil suction values as developed by the Great Britain Road Research Laboratory, but this system remains to be proved as a positive tool for the operating highway soils laboratory. There are also variations in the prediction of weight and amount of traffic as well as many other factors that the designer must assume at the time of designing the road. It is not the purpose of this paper to explore in detail all such variations but rather to emphasize those relating to construction. Nevertheless, it is necessary that such variations be considered in any evaluation of the effectiveness of a structural design system.

The contractor who builds a highway project has the obligation of furnishing and placing materials that comply with the specifications and plans. In so doing, he is required to place the materials in the structural section to certain specified thicknesses. Thickness would appear to be a noncontroversial, easy to obtain, and easy to measure specification. Yet, any inspector will confirm that this is not true. Materials paid for by the "square yard in place" tend to encourage keeping the thickness to a minimum. Materials paid for by the ton deposited on the grade have a reverse effect. On projects involving federal funds, layer thicknesses must be verified by cutting cores and digging holes in the completed structure. Table 2 gives summaries of thicknesses measured for various layers in the structural section in California from 1962 through 1969.

The ability of the contractor to accurately lay and place a layer is dependent somewhat on the accuracy required in placing the layer below. Therefore, the data given in Table 3 reveal an increasing degree of accuracy in the layer thicknesses from subbase through base to surface. This is fortunate because the lower materials are generally the cheaper materials. The figures on the asphalt concrete represent measurements that include surface, leveling, and base courses as one measurement. This includes all projects having between 0.2- and 0.6-in. total thickness. Although thicker

TABLE 2

THICKNESS MEASUREMENT VARIATIONS

Year	Material	Mean Deviation From Planned Thickness (ft)	Standard Deviation	Number of Measurements
1962	Asphalt concrete	+0.02	0.03	823
	Cement-treated base	+0.02	0.06	934
	Aggregate base	0.00	0.07	1,149
	Aggregate subbase	0.00	0.08	1,037
1963	Asphalt concrete	+0.01	0.03	1,327
	Cement-treated base	+0.02	0.06	1,173
	Aggregate base	0.00	0.06	1,310
	Aggregate subbase	0.00	0.09	1,183
1964-1965	Asphalt concrete	+0.02	0.03	1,760
	Cement-treated base	+0.02	0.05	2,187
	Aggregate base	0.00	0.06	1,285
	Aggregate subbase	+0.02	0.10	1,922
1966	Asphalt concrete	+0.02	0.04	1,569
	Cement-treated base	0.00	0.06	1,569
	Aggregate base	0.00	0.07	1,272
	Aggregate subbase	+0.03	0.12	1,833
1967	Asphalt concrete	+0.01	0.03	1,838
	Cement-treated base	0.00	0.06	1,412
	Aggregate base	+0.01	0.07	1,134
	Aggregate subbase	+0.03	0.11	1,887
1968	Asphalt concrete	+0.02	0.04	1,135
	Cement-treated base	+0.01	0.05	1,156
	Aggregate base	+0.01	0.06	828
	Aggregate subbase	+0.01	0.10	1,526
1969	Asphalt concrete	+0.02	0.04	1,323
	Cement-treated base	+0.01	0.06	1,318
	Aggregate base	+0.02	0.07	1,075
	Aggregate subbase	+0.02	0.11	1,370

TABLE 3

VARIANCE DATA FOR BASE AND SUBBASE MATERIALS

Test	Arithmetic Mean	Material Variance	Sampling Variance	Testing and Splitting Variance	Overall Variance	Overall Standard Deviation
Base Materials[a]						
Project B-1						
R-value	81.9	0.081	0.160	1.480	1.721	1.31
Sand equivalent	42.9	10.685	0.875	4.225	15.785	3.97
Percent passing No. 4	50.9	9.246	0.270	0.335	9.851	3.14
Percent passing No. 30	23.8	4.478	0.235	1.525	6.238	2.50
Percent passing No. 200	6.0	0.231	0.035	0.180	0.446	0.67
Project B-2						
R-value	79.9	1.133	0.0	4.695	5.828	2.42
Sand equivalent	30.6	35.171	0.525	1.325	37.021	6.08
Percent passing No. 4	58.1	5.603	0.700	1.710	8.013	2.83
Percent passing No. 30	27.3	4.402	0.400	0.580	5.382	2.32
Percent passing No. 200	7.9	0.952	0.075	0.215	1.242	1.11
Project B-3						
R-value	79.7	0.242	0.210	1.770	2.222	1.49
Sand equivalent	59.2	11.121	0.0	4.670	15.791	3.97
Percent passing No. 4	52.7	21.382	6.885	3.970	32.237	5.68
Percent passing No. 30	23.4	5.178	1.595	1.720	8.493	2.91
Percent passing No. 200	4.6	0.353	0.0	0.540	0.893	0.94
Subbase Materials[a]						
Project S-1						
R-value	68.8	14.612	0.0	25.855	40.467	6.36
Sand equivalent	30.2	3.502	0.0	12.835	16.337	4.04
Percent passing No. 4	49.5	14.378	0.265	3.685	18.328	4.28
Percent passing No. 200	7.8	0.474	0.190	1.100	1.764	1.33
Project S-2						
R-value	77.2	4.456	0.059	5.277	9.792	3.13
Sand equivalent	36.2	60.623	2.356	9.362	72.341	8.50
Percent passing No. 4	72.6	36.737	0.048	5.910	42.695	6.53
Percent passing No. 200	10.0	2.448	0.0	0.755	3.203	1.79
Project S-3						
R-value	70.9	54.038	0.0	25.250	79.288	8.9
Sand equivalent	29.2	5.519	0.0	1.915	7.434	2.73
Percent passing No. 4	45.0	34.253	3.275	5.990	43.518	6.60
Percent passing No. 200	8.6	2.245	0.110	0.450	2.805	1.68

[a]Slight negative variances were equated to zero.

layers have been placed on relatively few full-depth projects, a significant number have not yet been cored and measured. However, because the tolerance for making subgrade on foundation materials has not changed, it is anticipated that the standard deviations of full-depth asphalt pavements placed directly on the subgrade will increase and may approach those now recorded for the base materials.

In general, it would appear that variations in thickness of the asphalt concrete are more significant from a load-carrying viewpoint in thin layers than they are or will be in the thicker layers.

The strength of the various layers in the structural section is affected by compaction; and, in general, the higher the relative compaction is, the greater the strength is. Because of the difficulty in time required to make normal density and relative compaction tests, many states have adopted prescription types of compaction operations for the granular base and cement-treated base layers. Whereas this has some contractual advantages, it does not always result in the best possible compaction because granular materials can vary in their resistance to compaction. Sand, for instance, is usually difficult to compact, and graded aggregates are very easy. There are also differences between crushed and uncrushed aggregates. Several states are exploring and some specifying the use of statistical parameters to determine relative compaction.

The ability of the base materials to prevent lateral deformation within themselves and within the supporting basement soil has been determined by different agencies by using such tests as the CBR, R-value, Texas triaxial, and others. All of these tests are more or less empirical. However, each has proved its usefulness over the years. The principal disadvantage of such tests lies in the time required to perform them. At the speed of today's construction, the contractor can be far down the road before he learns that the material he placed a week ago does not comply with the CBR or R-value test.

Other tests in more or less general use as criteria of quality are gradation, the Los Angeles rattler, plastic index, sand equivalent, compressive strength for cement-treated base, percentage of crushed material, and petrological examination or durability test that will prevent the use of materials that disintegrate under adverse environments.

Since 1964 when the Bureau of Public Roads first encouraged research in measuring the variability of materials, there have been many reports on this subject. A good general coverage can be found in the six reports published during 1969 (2, 3, 4, 5, 6, 7). An attempt was made in these studies to separate material, sampling, and testing variances. Although the method used led to some doubts about its precise validity, it produced the first such information available to the highway engineer. Table 3 gives the results obtained on three projects in California (8). Generally, the data show small but significant variance due to sampling and larger variances in the material and testing areas. For these projects subbase material was obtained unprocessed from pits, and the base material was a crushed product processed by the contractor. As expected, the material variance for the subbase is high. However, the disturbing factor is that the testing variance increased in proportion to the material variance. When comparing these results to the base tests, one would conclude that the variance in the R-value test, for example, is not a uniform variance but is dependent on the type of material being tested. Such does not seem to be the case with the sand equivalent test, inasmuch as the results for base materials show overall standard deviations in the same ranges as those found for subbase materials.

Not one of the projects studied was in 100 percent compliance with the specifications, and yet the construction seemed to be normal, the materials were normal, and all the processes used were good. This led to conclusions, confirmed by other job tests, that judgment factors were being applied by field forces. This, in effect, made acceptance dependent on the amount of experience, background, and judgment of the inspector in charge of a particular operation. The final result of these studies has been the revision of specifications to include the opportunity for reasonable variability based on historical data of completed projects.

Specification tolerances and controls for the asphalt concrete layer of the pavement structure are generally tighter than those for the base and subbase layers. Nevertheless, the materials and workmanship that go into this layer are also subject to variations of significant magnitude. Granley (5) very capably describes variations in asphalt concrete construction. Table 4 gives a summary of 22 projects that were studied under the Bureau of Public Roads research program and reported by Granley. The data are similar to those given in Table 3 for the base and subbase materials, although they are more representative because they cover a wider geographical area. The data given in the table again confirm that variance is not a constant and that the sampling and testing errors are of significance.

Data reported for these projects also showed variations in asphalt content with a standard deviation of 0.28 percent for 23 surface course projects. The testing variance represented 40 percent, sampling variance 10 percent, and material variance 50 percent of the total variance. On the six base or binder course projects, the standard deviation was 0.35. The testing variance was 43 percent, the sampling variance 23 percent, and the materials variance 34 percent of the total. These variances indicate a need for more accurate testing and sampling procedures if the variations of the material are to be accurately determined.

Assuming proper mix design and satisfactory, available aggregates, we are still lacking a third and very important ingredient in asphalt concrete that affects the durability of the layer. This, of course, is the compaction of the hot mixture. Many articles have been written about the compaction of asphalt concrete, and all seem to emphasize that the mixture must be spread and compacted at the proper temperature to achieve maximum

TABLE 4

AVERAGES OF AGGREGATE GRADATION DATA FROM EXTRACTION TESTS

Sieve Size	Average Standard Deviation of Percent Passing	Shift of Mean From Job Mix Target	Average Variance as Percent of Total			Computed Average Compliance of Job Mix Tolerances
			Testing	Sampling	Material	
Surface Mixes (22 Projects)						
¾ or ½ in.	1.43	1.70	72	4	24	99
⅜ in.	2.49	1.73	29	31	40	93
No. 4	3.51	2.95	12	18	70	78
No. 8 or 10	2.81	2.45	10	15	75	77
No. 20 or 30	1.74	2.10	13	18	69	87
No. 40 or 50	1.37	1.72	18	15	67	87
No. 80 or 100	1.00	1.44	17	11	72	82
No. 200	0.94	1.43	21	14	65	74
Average	1.91	1.94	24	16	60	85
Base or Binder Mixes (6 Projects)						
¾ or ½ in.	4.33	1.66	65	13	22	83
⅜ in.	4.93	5.88	55	30	15	60
No. 4	3.92	2.03	46	17	37	76
No. 8 or 10	2.53	1.81	19	13	68	50
No. 20 or 30	2.17	2.22	25	28	47	81
No. 40 or 50	1.67	1.63	23	31	46	84
No. 80 or 100	1.15	1.23	30	30	40	97
No. 200	0.88	1.02	21	14	65	74
Average	2.70	2.19	36	21	43	76

density. Kilpatrick and McQuate (9) concluded that break-down rolling, either steel or pneumatic, should be concluded before the pavement temperature drops below 220 F. Experience in California tends to confirm this conclusion. For adequate compaction, the rolling pattern and the number of rollers required are dependent on the air temperature at which the mixture is placed, the thickness of layer, and the production of the contractor. Generally speaking, the thicker the layer is, the longer the allowable time cycle between the beginning of compaction and the end of compaction is. Stated another way, the proper compaction is achieved more easily and more readily in thicker lifts than in thinner lifts simply because the cooling of the mass is greatly retarded in thicker lifts.

It is not possible in this paper to explore all of the variations present in asphalt concrete mixtures. References previously quoted give a more complete picture. The discussion would not be complete, however, without covering the variability of in situ test data from tests on surfaces that have been used for a period of several years.

An article recently published by Welborn (10) discusses the durability factors of a group of projects constructed from 1954 to 1956. Table 5, taken from Welborn's report, gives the test variability after 10 to 12 years of service for six pavements all constructed with the same asphalt. Although there is a large spread in standard deviation. Welborn relates these variations very effectively to the void content of the mixtures. These are factors, however, that might be present in many asphalt concrete layers. How is the designer to anticipate the hardening that takes place in varying degrees? How can theory take this into account? What factors of safety are needed to prevent premature distress of the road surface? All of these questions and many more are in need of answers if a design method is to be adequately evaluated.

It is obviously impossible in a short paper such as this to discuss in any great depth all of the variations and the many ramifications found in variance of materials used in the highway structure. At best, only a few examples could be documented and reported.

It is extremely encouraging that a considerable amount of research effort has been expended in the past few years in measuring the variances of materials and construction operations. There is still much to be done. The designer needs better information on the anticipated in situ characteristics of foundation soils. He needs to have some method of determining the overall effect of variations in thicknesses in the various layers enter-

TABLE 5
VARIABILITY OF TEST DATA FOR PROPERTIES OF PAVEMENT SAMPLES

Property	Project Number	Variability			
		Minimum	Maximum	Mean	Standard Deviation
Penetration at 77 F	17	22	50	31.5	9.6
	18	31	70	47.6	14.4
	26	18	24	21.3	2.3
	27	23	74	46.7	17.6
	40	29	39	35.0	4.3
	41	17	38	23.8	7.3
Viscosity at 140 F, kilopoises	17	3.9	36.7	19.0	12.7
	18	2.5	17.0	9.1	5.6
	26	35.5	60.0	46.8	7.9
	27	2.1	32.0	11.6	11.2
	40	7.0	12.7	9.4	2.4
	41	13.4	315.2	119.0	104.0
Air void content, percent	17	0.1	4.9	2.5	1.5
	18	1.0	2.9	1.8	0.8
	26	4.4	7.2	5.7	0.9
	27	1.0	5.3	2.3	1.5
	40	2.1	2.9	2.4	0.4
	41	2.7	12.1	6.9	3.0
Voids filled with asphalt, percent	17	75.4	99.5	86.8	7.5
	18	84.1	94.0	89.4	4.3
	26	64.3	75.7	70.0	3.7
	27	73.4	93.3	87.4	6.8
	40	82.5	87.5	85.6	2.2
	41	47.2	85.0	67.5	12.1

ing in the structural section. He needs to have some assurance that construction methods are adequate and will result in a product equal to or better than the design requirements. He needs methods for estimating the detrimental effects of environment, which lead to changed properties of materials. Finally, he needs better methods of evaluating the product he designs because performance is the final measure of a good design.

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PART IV

IMPLICATIONS FOR FUTURE ACTIVITIES

FEDERAL HIGHWAY ADMINISTRATION

Charles F. Scheffey

In considering the discussion that has developed on what research is now most urgent to permit the establishment of a mechanistic approach to pavement design, I am painfully aware that we of the Office of Research have a great deal of work to do to respond to the workshop recommendations. I am also optimistic that we can extract from the thinking developed at the workshop a better plan for redirection of the pavement research program and, from the present program, specific items that are ready for implementation.

First of all, we will need to correct certain obvious gaps in communication, for it was apparent that important segments of the research program were unknown by most of the workshop participants. Important studies of traffic trends, truck size and weight, performance of documented pavement sections, developments in quality control and durability of materials, pavement roughness criteria, effects of roughness on heavy vehicle dynamics, and costs of management of highway maintenance are in progress. In our response to the report of this workshop, we will identify these studies and relate them to the program of recommended research.

Second, we will give prompt attention to the immediate revision of our plans for commitment of contract funds allocated to this program in the present fiscal year. We will also rework our entire research plan to ensure that the clearly indicated and documented priorities of this workshop are incorporated into our future plans in the most effective manner. Vigorous efforts will be made to ensure that all essential studies are initiated in a time frame that will produce interim procedures that permit the incorporation of rational techniques for prevention of specific types of pavement distress within the next few years. The program will also be arranged to lay a firm base for the establishment of a full system concept of pavement design. I am confident that the program can be funded through a combination of federal-aid research studies by the states, the NCHRP Program, other agencies, and our staff.

I should like to make some personal observations concerning the systems approach to pavement design. This label has been so overworked and misused lately that it is not always certain what is meant by the term. In the sense that it was used at this workshop, I think it focused our attention on the important tasks of defining performance criteria for pavements in a quantitative fashion and on the necessity of looking at the economics of pavements in the total context, which includes the costs of maintaining a specified level of service. I do, however, have reservations with regard to how far the systems analysis can be carried at the present time based on our knowledge of the estimating relationships and transfer functions for some of the vital subsystems. It is, therefore, my opinion that this activity should be continued at about its present level of effort until such time as we clarify some of the subsystem relationships.

Finally, I should like to take exception to the implication that there has been no prior effort to plan a concerted attack on the problem of coordinated planning of this research. The Office of Research and Development issued the task and study statements for the National Program of Highway Research in 1965. In that program there were several projects that laid out plans for reaching essentially all of the goals we discussed at this workshop. In particular, "Properties of Pavement System Components" called for research on materials characterization, including both primary response and flaw and fracture properties; and "Optimum Design of Structural Systems" included both the concepts of a systems approach to pavement design and a specific task on a rational design of flexible pavements. We have been moderately successful in the promotion of this program, although limits on our contract funds have prevented us from reaching the goals on the 5-year schedule originally projected. We are now in the process of revising and updating this program. This workshop was sponsored, in part, as a step in that process.

We cannot do the job alone. We view our mission as that of mobilizing the talent of the research community to reach the objectives desired. In a sense, it is not our program, but your program.

COMMITTEE ON THEORY OF PAVEMENT DESIGN

W. Ronald Hudson

This workshop was interesting and educational and provided an opportunity for a great deal of communication among interested parties in the pavement design field. It has also provided some perspective for the interaction of Highway Research Board committees interested in pavements. I will not belabor the general value of the workshop, inasmuch as that is covered elsewhere in this report. The fact that the workshop ultimately grew into discussions among members of the Committees on Theory of Pavement Design, Strength and Deformation Characteristics of Pavement Sections, and Mechanics of Earth Masses and Layered Systems is significant because the implication of the workshop should be to provide avenues of better communication and joint activities among these soils and design committees.

The teamwork involved in developing and putting on this workshop is illustrative of one of the major implications of the findings reported. Namely, a concerted team effort is needed to attack and solve successfully the complex problem of designing and managing pavement systems. If any agency or committee chooses to isolate itself from other agencies with related interests and interests involving other portions of the pavement system, it largely loses its effectiveness in performing its assigned or chosen tasks. Workshop Group C considered the overall design problem at some length and ultimately agreed on a conceptual model of the pavement design process as shown in Figure 3 of the paper by Hudson in this report. This was agreed to by the Committee on Theory of Pavement Design and, generally, other committees sponsoring the workshop.

Likewise, workshop Group I established unanimously that the pavement serviceability-performance concept typified by the AASHO Road Test PSI methods are at present the only known method of specifying and quantifying pavement performance and thus pavement failure. Certainly improvements are needed in any existing approach that uses this concept; however, the concept is basically valid. The other factor that was brought out at the workshop and that seems basic to pavement design is that a working pavement design system that incorporates as many of the parameters and factors into its models as possible is needed in order to provide at least a cursory basis for a sensitivity analysis comparing the effect of such parameters. Such a working system can more properly be called a pavement management system as suggested by several participants in the workshop, including Pister in his keynote address. No committee or section of the Highway Research Board is concerning itself with this important overall concept at the present time except perhaps the Committee on Theory of Pavement Design.

There was a strong overtone at this workshop of the importance of a mechanistically rational, theoretically based pavement structural subsystem, that is, one that uses proper materials characterization, proper formulation of boundary value problems, and proper solution of these problems to determine pavement behavior and proper distress analysis to determine limiting behavior or distress. However, it should be pointed out that the other two sponsoring committees are primarily concerned with this portion of the problem. The Committee on Strength and Deformation of Pavement Section is in fact concerned with realistic materials characterization as well as measurement of pavement behavioral characteristics. The Committee on Stress Analysis in Earth Masses is concerned primarily with the solution of boundary value problems and thus the theoretical prediction of stress, strain, and deflection.

Therefore, it seems obvious that the Committee on Theory of Pavement Design can best concern itself with the overall pavement management concept and with the integration of interrelated factors and design subsystems to obtain a useful pavement management system.

It is vital that this coordination and leadership be provided, and no other committee within the Highway Research Board committee structure performs this function. Nor is it handled in the national highway pavement and materials research program or among the many agencies involved.

COMMITTEE ON STRENGTH AND DEFORMATION CHARACTERISTICS OF PAVEMENT SECTIONS

John A. Deacon

As this workshop was originally conceived, its objective was a simple one: to develop or facilitate an interaction among three Highway Research Board committees that are involved with the improved design of pavement structures. Participation was subsequently expanded to include people who were not members of the three committees and who had some special interest in pavement problems or some special talent with respect thereto.

I am of the opinion that the extended, and sometimes heated, conversations at this workshop are evidence that the interaction objective was accomplished at least for the interim. Furthermore, I was impressed that the conversation included within it a great deal of communication inasmuch as each of us had an opportunity to be heard and each was listened to and, for the most part, understood.

Two other workshop objectives were subsequently added: to assess the current state of knowledge concerning pavement analysis and design and to list by priority the relevant research needs. We have largely accomplished these objectives, although, of course, we do not have complete accord among ourselves that the identification, documentation, and ranking of this information is optimal.

I would conclude, in any case, that regardless of whatever happens beyond this point the preliminary objectives of the workshop have been satisfied. However, we all hope that there may be additional continuing beneficial interaction as it concerns improved design and analysis of pavement systems.

It is interesting and perhaps necessary to conjecture what implications this workshop may have on the future activities of individual HRB committees. To examine this question requires that we first identify the functions of an HRB committee. Five of the seven stated functions of these committees relate to the initiation, conduct, evaluation, or implementation of research activities. Thus, the research function would seem to constitute almost the sole justification for the existence of these committees.

It is unfortunate, however, that the role is a passive one in that committees act largely in indirect ways rather than through direct participation and involvement as funding and contract agencies. In my opinion the most significant implications of this workshop will not be those for HRB committees but those for funding and contract agencies or for each of us as individual design professionals.

However, there are some direct implications for future committee activities, and let me briefly explore those that are possibly relevant for the Committee on Strength and Deformation Characteristics of Pavement Sections.

SCOPE OF COMMITTEE

There has been no indication that a shift in role or scope of this committee is required. Our role is reasonably well defined, with the possible exception of evaluating the response of in situ pavements to traffic loads, and any overlap with the activities of other committees is minimal.

INTERACTION WITH OTHER COMMITTEES

The nature of the pavement system is such that continued interaction among all HRB committees concerned with pavement analysis and design is essential. For example, it is apparent that the technical aspects of nonlinear characterization will require a coordinated approach between those responsible for testing and those responsible for analysis. Furthermore, the research need to "determine applicability of linear theories to predict stresses, strains, deflections, and fatigue distress in pavements" will doubtlessly require close technical coordination and interaction. As a first step toward ensuring continued interaction, liaison representatives to other related committees have been appointed. However, additional means for ensuring that our heretofore disjointed activities are better integrated into a team effort need to be explored.

GUIDANCE FOR COMMITTEE DECISIONS

There is little doubt that the deliberations of the workshop will exert a profound influence on the thinking of the committee as we approach the tasks of further identifying our functions and planning for future activities. Several decisions currently facing the committee will be resolved in part through the broadened perspective created by the workshop.

DATA POLLUTION

One recurring thought that permeated the workshop was that a rather large storehouse of information exists that has yet to be assimilated in a meaningful way. One workshop attendee referred to this as "data pollution," a term connoting an undesirably high level or concentration of data. Our committee should try to ascertain if such dangerously high levels of unused or unavailable data exist and, if so, should try to take some action in this regard.

RESEARCH THAT CAN BE IMPLEMENTED

A number of subsystems within the pavement design framework that are now "technologically implemented" were identified at the workshop. The problem is, however, that these subsystems may not be "practically implemented" in that they may not be generally available to interested designers and may necessitate underlying assumptions about which designers have little knowledge. The HRB committees could make a significant contribution by explicitly defining the subsystems that can be implemented and by preparing directions for their implementation. A good illustration is the iterative, quasi-linear elastic analysis used by Monismith and others to estimate stress, strain, and displacement states within a pavement. The general consensus of workshop Group A participants was that this approach has a great deal of validity and utility. However, it requires that complex material behavior be approximated by a modulus and by Poisson's ratio. How best to obtain this characterization is a question that could well be answered by our committee in the form of interim guides that represent our best composite knowledge to date.

RESEARCH NEEDS

Also identified by the workshop participants was a list of research needs. The needs statements were by necessity most general, however, and their utility could be improved by additional definition and elucidation. Because the identification of research needs is one of the primary functions of HRB committees, our committee could provide a useful service in this regard.

INFORMATION GAP

The workshop illustrated once again that rather large information gaps still exist among the many groups and individuals of our profession. It is incumbent on all HRB committees to search continually for effective means of bridging these gaps.

COMMITTEE ON MECHANICS OF EARTH MASSES AND LAYERED SYSTEMS

Russell A. Westmann

The scope of the Committee on Mechanics of Earth Masses and Layered Systems includes research and investigative studies related to theory and mechanics of the behavior of earth masses and layered systems. This includes the applications of elasticity, viscoelasticity, plasticity, and consolidation, as well as certain aspects of computer science, to predict behavior of the earth masses and layered systems. In addition, the committee's scope includes those studies that test the validity and accuracy of existing theories and methodologies under field conditions. The following comments are intended for anyone with interests and activities aligned with the committee's scope.

The workshop has clearly stated the pavement design problem in the context of a systems framework. This in turn has forced the committee to closely examine its role in the design process and squarely face its specific responsibilities in the overall system.

This role is primarily as follows: If the analyst is given the inputs of load and environment (or their statistics) as well as the proposed geometry and material components and their mechanical characterization, then he is asked to make the best possible prediction of the indicators of distress. In particular, it is desired that he estimate the permanent deformation, the appearance of fatigue cracking, and the propagation of existing cracks. General information the designer typically wants includes the stress and strain states and the displacement fields throughout the layered system.

Any activity that helps this committee to fulfill this role and improve the predictive capability outlined is justified. Otherwise, the activity is questionable as far as furthering pavement design is concerned.

In my opinion, committee activities pertinent to the design process are as follows.

IMPROVEMENT ACCESSIBILITY

The committee must encourage the continued availability of usable, well-documented, and properly maintained computer programs representing the most up-to-date prediction algorithms. In particular, effort must be immediately made to make the linear elastic and viscoelastic layered system theory available to the highway engineer through appropriate computer programs.

Because the designer wishes to know the stress, strain, and displacement fields throughout the system, it is necessary to present all of this information in such a way that it can be rapidly evaluated. One way of achieving this is by computer graphical display. Of course, other forms of presentation such as tables, charts, and graphs should not be overlooked.

ASSESSMENT OF METHODS OF PREDICTION

The committee must encourage and help with the implementation of the correlation of the best prediction methods with controlled and well-instrumented field tests. At the present time this means that the accuracy of the linear elastic and viscoelastic layered system theories must be assessed. In making such an assessment, the committee must interact with highway engineers.

EXTENSION AND DEVELOPMENT OF PREDICTION METHODS

In the event a prediction method is not sufficiently accurate, it becomes necessary to make appropriate modifications. A short-term modification might consist of certain engineering or ad hoc adjustments so that the predictions better fit the field results. For example, the linear elastic theory might be altered in an approximate manner to permit the prediction of the permanent surface deformations.

A longer range approach is to develop a more sophisticated theory by accounting for the material properties in a more realistic and accurate manner. Because the key point here is the mechanical characterization, members of the committee must be prepared to advise the materials group with respect to appropriate stress-strain-temperature-time relations as well as to assist with the solution to any boundary value problems pertinent to the test configurations. In characterizing the material, attention must continually be focused on the distress mechanisms of permanent distortion and fracture arising from environment and wheel loads.

Once the material characterization is completed, the improved prediction method can be developed by solving the appropriate boundary value problem. This new theory must then be assessed as before, and, if successful, the results must be made accessible to the highway engineer.

SPECIFIC PROBLEMS

Specific problems that need immediate attention include the following:

1. Stochastic analyses,
2. Analysis of the reflection problem,
3. Shoulder analysis,
4. Interaction of distress mechanisms, and
5. Pavement response to environmental inputs.

Until the present time surprisingly little has been done to determine the statistical response of a layered system due to a statistical distribution of loadings or for a nondeterministic set of material properties. This aspect of the analysis must be considered if the prediction algorithm is to fit into the overall design system.

In the future it is expected that the reflection cracking of pavements will be an important problem. At the present time even the simplest models do not exist, and the basic reflection mechanism is not understood. Another problem deserving attention concerns the effect of the finite width of the pavement or the presence of the shoulder of the road.

The interaction of distress mechanisms is an important topic not previously considered. For example, what is the effect of an existing crack on further crack development? Or, if permanent deformation leads to an uneven surface, then is the effective wheel load increased and further pavement distress hastened?

Finally, the problem of determining the response of layered systems subjected to environmental inputs has received little attention. Questions of just how thermal and moisture changes affect the pavement remain to be completely answered.

These are some of the more important problems that deserve the committee's immediate attention. To obtain complete answers to these questions requires close teamwork among the highway engineer, designer, materials expert, and analyst. Only in this way can a design answer be obtained that is acceptable within the systems framework presented at this workshop.

PART V

PARTICIPANTS, ACKNOWLEDGMENTS, AND SPONSORING COMMITTEES

PARTICIPANTS

Ernest J. Barenberg
Department of Civil Engineering
University of Illinois
Urbana

Richard D. Barksdale
School of Civil Engineering
Georgia Institute of Technology
Atlanta

Arthur T. Bergan
University of Saskatchewan
Saskatoon, Saskatchewan, Canada

James Brown
Texas Highway Department
Austin

John E. Burke
Illinois Division of Highways
Springfield

W. N. Carey, Jr.
Highway Research Board
Washington, D. C.

John A. Deacon
Department of Civil Engineering
University of Kentucky

J. C. Dingwall
Texas Highway Department
Austin

Fred N. Finn
The Asphalt Institute
Berkeley, California

J. E. Fitzgerald
Civil Engineering Department
University of Utah
Salt Lake City

Earnest F. Gloyna
College of Engineering
University of Texas
Austin

John M. Griffith
The Asphalt Institute
College Park, Maryland

J. W. Guinnee
Highway Research Board
Washington, D. C.

Ralph C. G. Haas
Department of Civil Engineering
University of Waterloo
Waterloo, Ontario, Canada

C. R. Hanes
Ohio Department of Highways
Columbus

Milton E. Harr
Department of Civil Engineering
Purdue University
Lafeyette, Indiana

James H. Havens
Kentucky Department of Highways
Lexington

John W. Hewett
Highway Design Division
Office of Engineering
Federal Highway Administration
U. S. Department of Transportation
Washington, D. C.

James M. Hoover
Department of Civil Engineering
Engineering Research Institute Laboratory
Iowa State University
Ames

W. Ronald Hudson
Department of Civil Engineering
University of Texas
Austin

Donald A. Kasianchuk
Carleton University
Ottawa, Ontario, Canada

Wolfgang G. Knauss
California Institute of Technology
Pasadena

Clyde N. Laughter
Wisconsin Department of Transportation
Madison

Roger V. LeClerc
Washington State Department of Highways
Olympia

Clyde E. Lee
Bureau of Engineering Research
University of Texas
Austin

Wallace J. Liddle
Utah State Department of Highways
Salt Lake City

James W. Lyon, Jr.
Louisiana Department of Highways
Baton Rouge

Kamran Majidzadeh
Ohio State University
Columbus

Alfred W. Mann
The Asphalt Institute
College Park, Maryland

Richard A. McComb
Structures and Applied Mechanics Division
Office of Research
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C.

B. F. McCullough
University of Texas
Austin

Chester McDowell
Texas Highway Department
Austin

Thurmul F. McMahon
Materials Division
Office of Research
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C.

Phillip L. Melville
Airport Design Section
Federal Aviation Administration
U.S. Department of Transportation
Washington, D.C.

Fred Moavenzadeh
Massachusetts Institute of Technology
Cambridge

Carl L. Monismith
Institute of Transportation and Traffic Engineering
University of California
Berkeley

Lionel T. Murray
Materials and Research
Missouri State Highway Department
Jefferson City

Keshavan Nair
Materials Research and Development, Inc.
Oakland, California

F. N. Nichols, Jr.
National Crushed Stone Association
Washington, D.C.

William H. Perloff
School of Engineering
Purdue University
Lafayette, Indiana

Karl S. Pister
University of California
Berkeley

James M. Rice
Materials Division
Office of Research
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C.

Charles F. Scheffey
Office of Research
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C.

Robert L. Schiffman
University of Colorado
Boulder

Frank H. Scrivner
Texas Transportation Institute
Texas A&M University
College Station

George B. Sherman
Materials and Research Department
California Division of Highways
Sacramento

James F. Shook
Special Projects Section
The Asphalt Institute
College Park, Maryland

Eugene L. Skok, Jr.
Department of Civil Engineering
University of Minnesota
Minneapolis

Avery Smith
Texas Highway Department
Austin

Harry A. Smith
National Cooperative Highway Research Program
Highway Research Board
Washington, D. C.

L. F. Spain
Highway Research Board
Washington, D. C.

Ronald L. Terrel
Department of Civil Engineering
University of Washington
Seattle

Bernard A. Vallerga
Materials Research and Development, Inc.
Oakland, California

N. K. Vaswani
Virginia Highway Research Council
Charlottesville

Aleksander S. Vesic
Department of Civil Engineering
Duke University
Durham, North Carolina

Harvey E. Wahls
Department of Civil Engineering
North Carolina State University
Raleigh

Russell A. Westmann
Department of Mechanics and Structures
University of California
Los Angeles

E. B. Wilkins
British Columbia Department of Highways
Victoria, British Columbia, Canada

Stuart Williams
Structures and Applied Mechanics Division
Office of Research
Federal Highway Administration
U. S. Department of Transportation
Washington, D. C.

Matthew W. Witczak
The Asphalt Institute
College Park, Maryland

Eldon J. Yoder
Purdue University
Lafayette, Indiana

S. R. Yoder
Indiana State Highway Commission
Indianapolis

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