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Application of Pavement Design Models

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Abridgment

Pavement Management: The Network-Level Decision Process

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The pavement management process is described in terms of two generalized management levels: the network level, involving global administrative decisions, and the project level, involving technical decisions for specific projects. A general framework for a comprehensive pavement management system (PMS) is described, based on the flow of information at either management level. Some essential features of pavement management systems are identified, and the network-level subsystems of the general PMS framework are discussed in detail. Specific network-level techniques that seem promising for use within a comprehensive PMS are identified. It is concluded that, although the development of network-level techniques has generally lagged behind project-level development, implementation of a comprehensive PMS can begin immediately by using existing technology.

Pavement management involves the coordination, scheduling, and accomplishment of all activities performed by a highway agency in the process of providing adequate pavements to serve the public. The systems approach to pavement management provides a rational, highly structured decision process with the objective of providing the highest possible value for public funds expended on pavements. This is accomplished by comparing investment alternatives (coordination of design, construction, maintenance, and evaluation activities) and making efficient use of existing methods and knowledge (1,2). The idea behind a pavement management system (PMS) is to improve the efficiency of the normal decision-making process, to expand its scope, to provide feedback as to the consequences of decisions, and to ensure the consistency of decisions made at different levels within the agency (3).

A PMS is a system that provides decision makers at all management levels with optimal strategies derived through clearly established rational procedures. Such a system provides an evaluation of alternative strategies over a specified analysis period based on predicted values of quantifiable pavement attributes and subject to predetermined criteria and constraints. A PMS involves an integrated, coordinated treatment of all areas of pavement management and is a dynamic process that incorporates feedback on the various attributes, criteria, and constraints included in the optimization procedure. Fuller discussion of the essential characteristics of a PMS and their secondary requirements is available elsewhere (3,4).

FRAMEWORK FOR PMS

The three major activity types of a comprehensive PMS are shown in Figure 1. Decision-making activities are characterized by their scope: wide-ranging or global decisions that affect the whole highway network and specific decisions on individual projects. The network-management level primarily involves programming and budgeting decisions for groups of projects or an entire network. The project-management level is characterized by predominantly technical management concerns on individual projects. Of course, technical activities occur at the network level, and administrative activities occur at the project level. It is necessary to make a distinction, however, because detailed technical decisions for individual projects involve considerations fundamentally different from the more global decisions that must be made for a network.

These two types of decision-making processes constitute virtually the entire system from the typical user's point of view. That is to say, the PMS is primarily a tool for use by decision makers. Other activities that provide feedback for updating components are vital to the proper functioning of the system but generally remain behind the scenes.

A total PMS functions at all management levels, but each level requires different types and amounts of data, uses different decision criteria, and operates under different constraints. Consequently, the detailed structure of the total system may vary considerably from level to level; however, the basic sequence of actions within levels is the same.

The flow of information within a comprehensive PMS framework is illustrated in Figure 2. First, pertinent information is gathered, and the consequences of the available choices are analyzed in the light of this information. Based on this analysis and on other nonquantifiable considerations (which may involve political and social issues), a decision is made. The decision is then implemented, and the results are recorded in the data base and passed on to other management levels.

This three-subsystem concept as applied to a two-level PMS is shown in Figure 3, as well as the relationships among the subsystems and with the data base. The result of a decision at the network level forms part of the input for the project-level decision process and vice versa. Thus, the information flow within such a management system is cyclical in nature. Not illustrated is the fact that a single decision at the network level often triggers multiple decisions for a large number of individual projects.

NETWORK-LEVEL SUBSYSTEMS AND MANAGEMENT ACTIVITIES

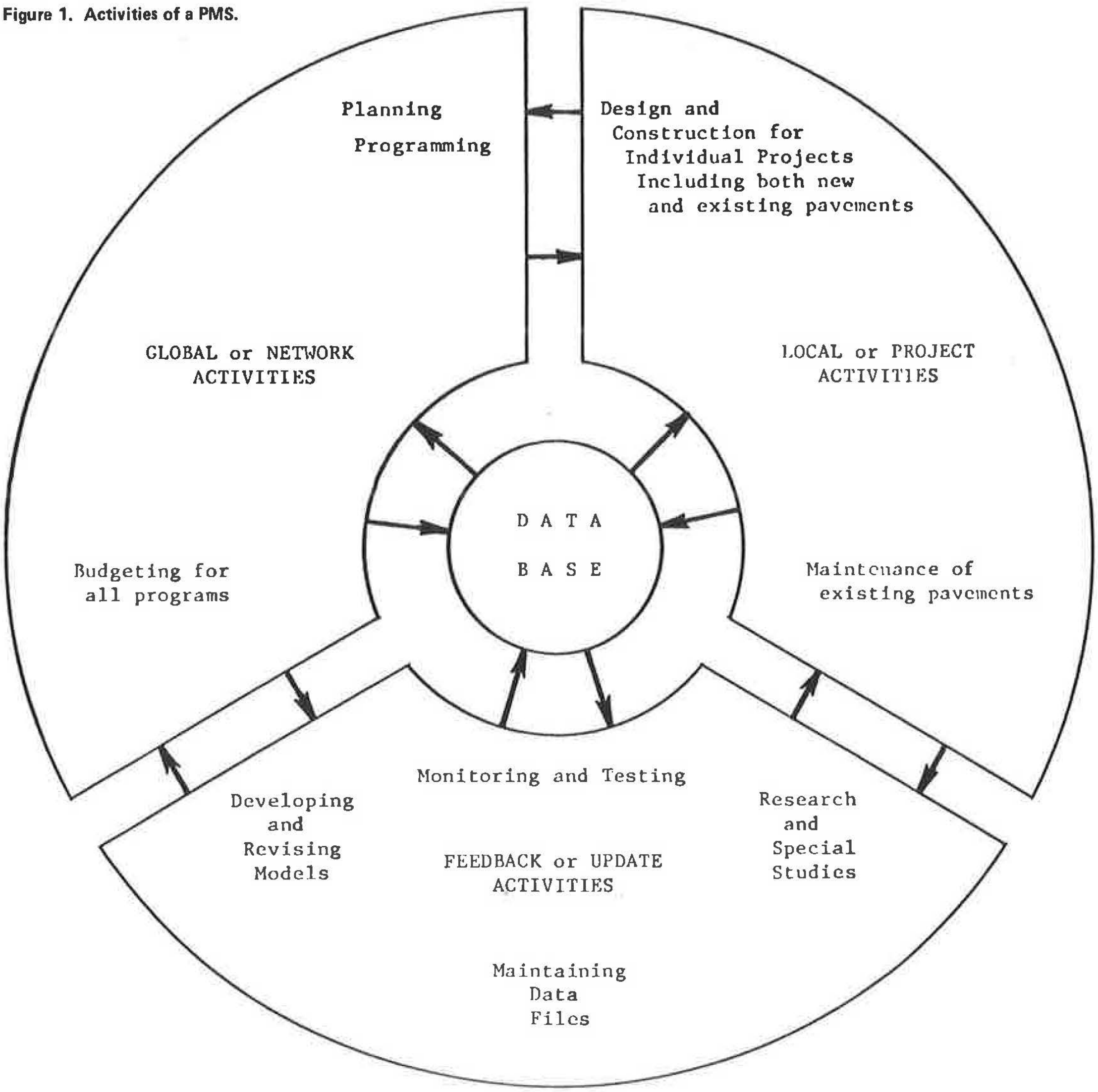
The network-level subsystems of Figure 3 are discussed briefly below. More-comprehensive discussion, including project-level activities, is available elsewhere (3,4).

Information Subsystem

The information subsystem contains information necessary to determine the condition of the network as a whole. The essential activities required to support this subsystem include

1. Determination of the pavement attributes to be measured, such as structural capacity, ride quality, surface condition, and skid resistance, to a degree of accuracy, intensity, and frequency appropriate to the class of roadway, agency resources, etc.;
2. Identification of homogeneous sections or links in the network;
3. Collection of nonpavement information such as geometry, traffic, and accidents;
4. Conducting of field measurement programs;
5. Generation of historical maintenance data and construction records;
6. Estimation of unit costs for materials and work;

Figure 1. Activities of a PMS.



7. Identification of available resources; and
8. Identification of already committed improvements.

Analysis Subsystem

The essential function of the analysis subsystem is to consider pavement improvement and maintenance needs and to arrive at a program of rehabilitation, new construction, and/or maintenance. This is accomplished through the following activities:

1. Identify needs and "candidates" for improvement;
2. Generate alternatives for each candidate project (i.e., several types, thicknesses, and timings for new construction, rehabilitation, or maintenance);
3. Select program-analysis period, discount rate, minimum ride-quality levels, etc., for technical and economic analysis;
4. Identify the basis for deciding on the final program, i.e., solely economic (maximization of benefits or minimization of costs) or partly economic and partly nonquantitative;
5. Estimate performance for each alternative;
6. Perform economic analysis for each alternative; and
7. Develop initial programs for construction, rehabilitation, and maintenance, optimized according to a selected measure of benefit or ranked by priority.

Implementation Subsystem

The implementation subsystem of the network-management level of Figure 3 includes accomplishment of the selected program of work. Major activities are

1. Identifying the recommended work program,
2. Submitting the program for legislative or administrative approval,
3. Budgeting and reporting of program to project-management levels,
4. Scheduling and accomplishing work,
5. Updating data records and prediction models periodically, and
6. Revising program as necessary.

EXISTING TECHNOLOGY AND THE GENERAL PMS FRAMEWORK

This framework provides a basis to compare and assess the current pavement management procedures

Figure 2. Flow of information at a typical management level within a total PMS.

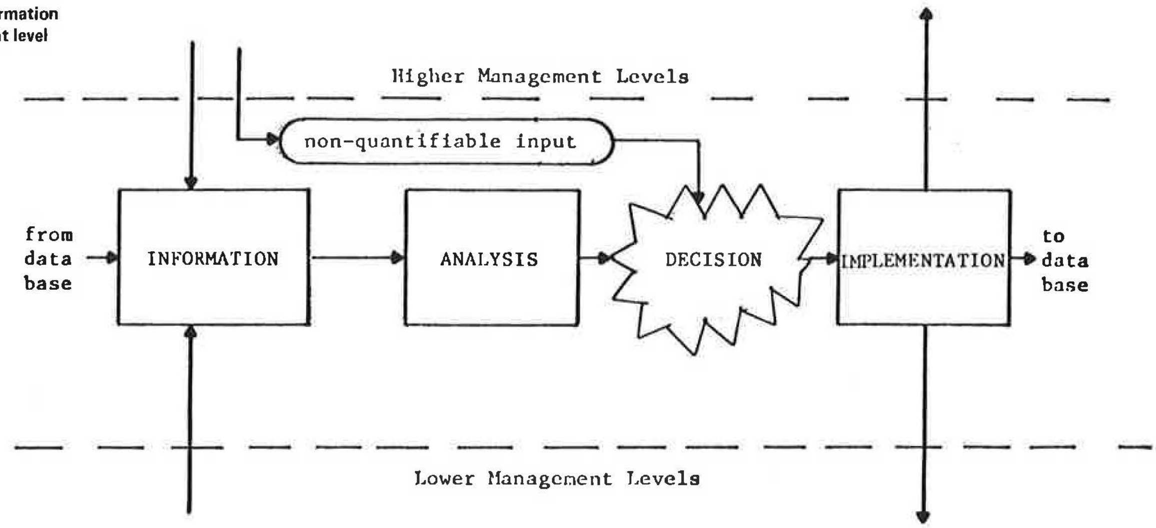
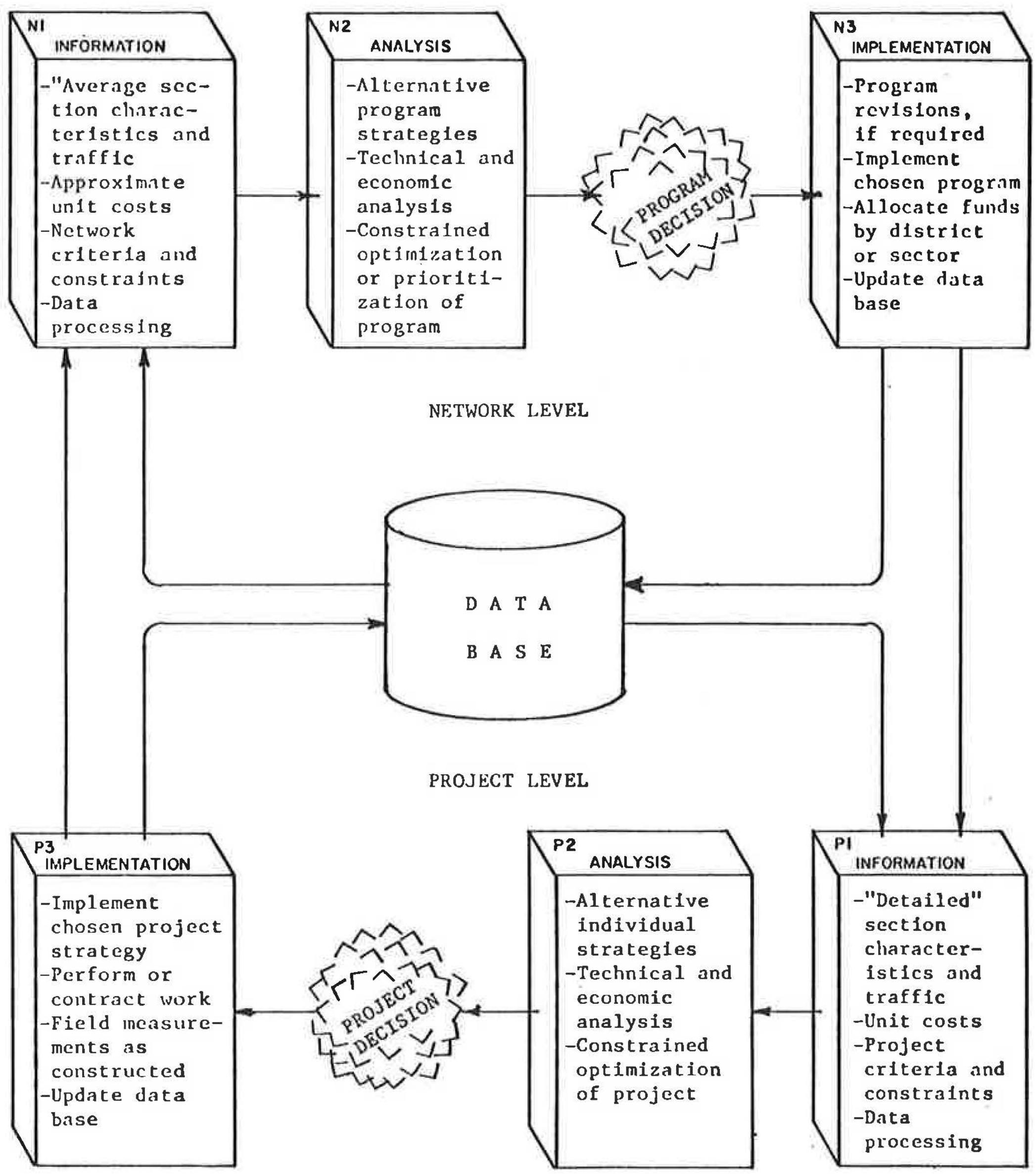


Figure 3. Framework and major subsystems of a two-level, comprehensive PMS.



used by various highway agencies. The Workshop on Pavement Management sponsored by the Federal Highway Administration and conducted in Tumwater (Olympia), Washington, in November 1977 provides a succinct statement of current practice for several states (5).

The techniques described at Tumwater are principally directed toward detailed technical decisions for individual projects. The network-level decisions that are made usually aggregate the project-level results to develop priorities in a program for rehabilitation. Such a process is not optimization of network alternatives. Most participating agencies recognized the need for systematic network-level decisions, and some are moving in that direction. Arizona, for example, is currently developing a network-level constrained optimization scheme to complement its existing project-level system (according to Fred N. Finn). Another example is the Rehabilitation and Maintenance Strategies (RAMS) program under development for the Texas State Department of Highways and Public Transportation by Lytton and others (6-8). In addition, some relative newcomers (such as Idaho) have made impressive starts, and considerable development and implementation has occurred in Canada (9-12). Finally, potentially useful network-level techniques have been reported by several other researchers (13-15).

Even though much of the work discussed at the Tumwater Workshop is project-oriented, several states use procedures that are appropriate for network-level use. A few of the more promising techniques are discussed below and placed within the framework of Figure 3.

Because of the broad scope of network-level decision making, it is nearly essential that a single variable or objective function be chosen for optimization. A strategy can then be selected to maximize (or minimize) the value of this single variable, subject to any applicable constraints. The most straightforward approach involves the choice of a directly measurable variable as an objective function.

This concept has been developed to a considerable extent by New York State. There the user is considered to be the final judge of the pavement. New York has developed a specialized measure of riding quality (related to the Present Serviceability Rating concept) that is applied to all pavement sections in the highway network. This measurement variable is used to set priorities for projects. The procedure involves physical-roughness measurements that are correlated to user ratings. The New York rating procedure also uses psychophysical scaling techniques in the determination of the rating score, large rating panels (from 60 to 80 people), and separate evaluations for flexible, rigid, and composite pavements.

A second approach to the use of a single objective function involves the combination of multiple variables to produce a single index. Arizona uses a combination of cracking, riding quality, deflection, age, traffic, environment, and functional class of roadway to produce a weighted average of total lane miles in acceptable condition over the specified analysis period. This combined-attribute approach guarantees the consideration of all important pavement variables in the optimization procedure. The single attribute, if carefully chosen, will reflect to some degree all of the multiple values cited above yet will be more efficiently and less expensively measured. The choice between single- or multiple-attribute objective functions depends on whether the agency can find a single variable that it considers a satisfactory measure of pavement adequacy.

Another important part of network-level analysis involves consideration of the relative costs of alternative strategies. The Arizona system considers a road's life-cycle costs to be constrained by some preset budget level. At the other extreme, the Washington State Department of Transportation uses the present value of total expected costs over a long-term analysis period as a parameter for optimization. Many states favor consideration of costs that include a budget constraint. For network-level analysis, evaluation of cost could be in a benefit/cost ratio form, just as for project-level analyses. The use of either a budget constraint or an optimization with respect to a benefit/cost ratio is compatible with the framework and concepts described in this paper. The choice is largely a matter of attitude and perspective of the implementing agency.

The network analysis could provide the ability to estimate the average network service level to be expected for a given budget and, conversely, given the desired network service level, to estimate the required budget. Once the procedure to calculate costs and service levels or benefits under various strategies has been established, both analyses can be provided on a routine basis and could be helpful in responding to various legislative inquiries.

SUMMARY

Clearly, no existing method represents a complete, working PMS as envisioned in Figure 3. Many agencies have, however, taken a good first step in achieving such a system, and a few systems currently in use or under development are well advanced. The development of a total management system must, in fact, proceed through many small steps so that each component may be tried and modified, new components may be properly interfaced, and a smooth transition may be effected in the operating procedures of the agency. Only after such a process will the management system be accepted and used by an agency.

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Simplified Structural Analyses of Flexible Pavements for Secondary Roads Based on ILLI-PAVE

JOSE L. FIGUEROA AND MARSHALL R. THOMPSON

A procedure based on the results of a stress-dependent finite-element computer model and used to calculate the resilient response parameters of conventional flexible pavements subjected to traffic loads is presented. Flexible pavements composed either of a granular base protected by a surface treatment or of an asphalt-concrete surface layer and a granular base are considered. Asphalt-concrete surface-layer thickness, base thickness, modulus of elasticity of the asphalt concrete, and subgrade resilient modulus at the break point (the input data required to calculate the resilient response parameters) are discussed. The basis of a flexible pavement design process for secondary roads is proposed.

This paper describes the development of a simplified procedure for the structural analysis of pavements for secondary roads. Preliminary research conducted by the Transportation Materials Engineering Group of the University of Illinois has indicated the feasibility of predicting critical pavement-response characteristics (stresses, strains, and deflections) by means of simple algorithms. Those algorithms have been reviewed and extended to include other parameters considered to play an important role in pavement response to loading.

A finite-element computer program for flexible pavement analysis originally developed by Wilson (1) and later modified by Duncan and others (2) and by Raad and Figueroa (3) served as the main research tool. The recent modifications introduced by Raad and Figueroa (3) provide a more rational assessment of the state of stress of pavement materials approaching failure (and consequently their moduli values) according to the Mohr-Coulomb theory of failure. The finite-element computer program for flexible pavement analysis (ILLI-PAVE) offers an alternate iterative method of solution in addition to the already known incremental procedure. Stress-dependent material models can be considered in this model. Traylor (4) showed that the ILLI-PAVE program adequately predicted the flexible pavement response to loading when the results of the computer modeling and field test data were compared.

The objective of this research was to develop a procedure that would eliminate the need to use a computer model for flexible pavement analysis every time the pavement response to loading was to be determined. The methodology consisted of the ILLI-PAVE analyses of pavements having all possible combinations of input parameters identified to be the best determinants of pavement response to loading. The parameters were chosen within the range of material properties and layer thicknesses expected for secondary roads. Multivariable regression analyses were performed on the ILLI-PAVE results to develop algorithms expressing each pavement response (dependent variable) in terms of material properties and geometric characteristics (independent variables) of the flexible pavement. A description of the assumed loading conditions, composition of pavement cross section, and range of material properties is given below.

DESCRIPTION

Loading Conditions

A constant load of 40 kN (9 kips) was maintained throughout the study to account for half of the 80-kN (18-kip) single-axle load commonly used for design. In the mathematical representation, the load was applied on top of the upper layer and uniformly distributed over a circular area with an intensity of 550 kPa (80 psi).

Pavement Cross Section

The common construction practice for secondary roads is to place a granular base protected by a surface treatment directly over a generally fine-grained subgrade. For higher traffic volume, a thin asphalt-concrete surface layer is sometimes added on top of the base.

Three asphalt-concrete layer thicknesses of 0.0 mm (representing a surface treatment), 38 mm (1.5 in), and 76 mm (3.0 in) were included in the analysis. Granular-base layer thicknesses of 102, 152, 229, and 305 mm (4, 6, 9, and 12 in) were used in the ILLI-PAVE analyses. The subgrade was a fine-grained soil limited to a depth of 50 times the radius of the loaded circular area measured from the pavement surface.

Material Properties

The ILLI-PAVE program required the definition of some material properties. Unit weight, Poisson's ratio, and modulus of elasticity for linearly elastic materials or resilient modulus model for stress-dependent materials need to be specified. In addition to these, the ILLI-PAVE program requires the earth-pressure coefficient at rest (Ko) and shear-strength characteristics of granular and fine-grained soils. A detailed discussion of the most pertinent material properties is given below and classified according to material type.

Asphaltic Materials

Based on a consideration of the temperature dependency of asphaltic materials and the range of pavement temperatures expected during the most critical period of subgrade support, the modulus of elasticity of the asphalt concrete was varied between 690 MPa (100 ksi) and 9650 MPa (1400 ksi). In addition to these two values, the effects of using an intermediate 3450-MPa (500-ksi) modulus of elasticity were examined.

Granular Materials

The resilient modulus of compacted granular soils (Er) can be expressed in terms of the sum of the three principal stresses (θ). Allen and Thompson (5) tested a wide variety of granular materials for the purpose of defining their resilient characteristics. The coefficients defining the resilient-modulus model varied with material type, density, and method of testing. Although Poisson's ratio was also found to be stress dependent, Allen and Thompson (5) concluded that a constant value in the range of 0.35-0.40 could be assigned to granular layers without sacrificing accuracy in the results. For the granular materials considered in this investigation, the Poisson's ratio was always equal to 0.38.

Traylor (4) tested granular materials used in the construction of the AASHO Road Test loops. After a series of tests were run, average models were determined. For dense-graded crushed limestone, the resilient-modulus model was best represented by the relationship

$$Er = 9000 \times \theta^{0.33} \qquad (1)$$

where Er = resilient modulus (psi) and θ = sum of principal stress (psi). This model was used throughout the analyses since it represents the behavior of a material commonly used in the construction of secondary roads. A 40° angle of shearing resistance was assumed to be reasonable for the crushed-stone base.

Fine-Grained Soils

Results from repeated-load testing of fine-grained soils indicate that the resilient behavior of these soils is highly dependent on the deviator stress. Extensive testing programs have also identified the soil properties that control the resilient characteristics of these soils.

Thompson and Robnett (6), after working with most of the soil series contained in the 26 soil-association areas in the state of Illinois, determined average resilient property data and the standard deviation of the scatter of data for soils grouped according to their moisture content. Statistical analyses were performed for soils at optimum moisture content, optimum moisture plus 1 percent, and optimum moisture plus 2 percent (AASHTO T99). The slopes of the two straight portions of the curve that defines the relation between the resilient modulus and the deviator stress (Figure 1), as well as the deviator stress at which the break point occurs, did not change appreciably with respect to the degree of saturation. Thus, a constant value could be assumed for these three parameters. The resilient modulus at the break point, however, changed significantly with the degree of saturation. It could be regarded as a vertical displacement of the resilient modulus versus deviator stress curve with respect to the degree of saturation.

Three of the resilient-modulus models for subgrade soils included in the computer analyses (stiff, medium, and soft models, as indicated in Figure 1) were chosen based on the work done by Thompson and Robnett (6). Robnett and Thompson (7) also studied the effects of high moisture content and freeze-thaw cycles on the resilient behavior of a number of fine-grained soils. In general, moisture variations and cyclic freeze-thaw action affected the resilient behavior of this type of soil. Appreciable softening was noticed when the moisture content increased or after a freeze-thaw cycle. Comparison of the weakest assumed resilient-modulus model (soft) with the quantitative results shown by Robnett and Thompson (7) indicated that an additional "very soft" resilient-modulus model needed to be assumed to account for those soils highly susceptible to high moisture and freeze-thaw cycling effects.

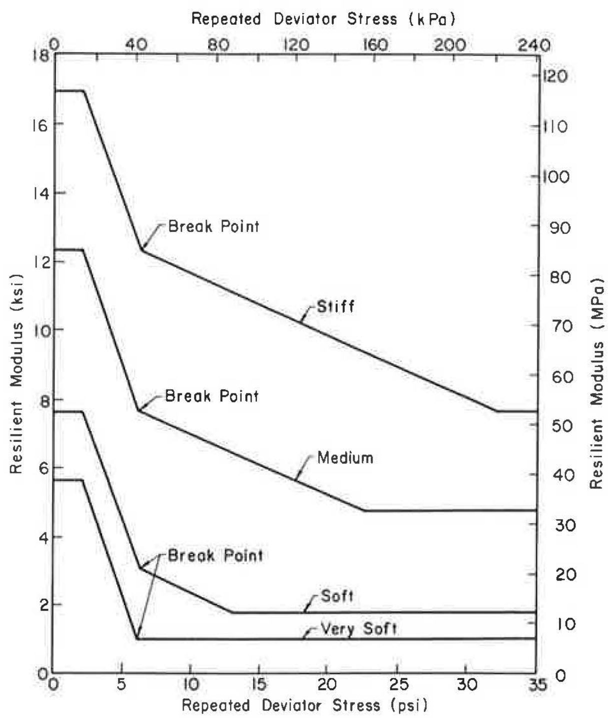
Figure 1 gives a complete representation of the subgrade material models used in the ILLI-PAVE analyses. A constant value between 0.45 and 0.49 is usually adequate to represent the Poisson's ratio of fine-grained soils in pavement structural analysis. Therefore, Poisson's ratio was not varied for the four types of fine-grained soils considered in this research; it was assumed to be equal to 0.45.

DEVELOPMENT OF ALGORITHMS REPRESENTING THE PAVEMENT RESPONSE TO LOADING

Resilient pavement response is primarily determined by loading conditions, material properties, and geometric characteristics of the pavement. For a given loading condition, the geometric characteristics and some of the material properties are basic parameters for the design of secondary road pavements, or they influence the pavement response throughout the life of the pavement. The asphalt-concrete surface-layer thickness (tac) and the granular base thickness (tbase) are the geometric characteristics that have the most influence on flexible pavement response to traffic loads.

The material properties that most influence the resilient response are the modulus of elasticity of the asphalt concrete (Eac), the resilient modulus of fine-grained soils at the break point (Eri) (the slopes of the straight lines on either side of the break point are approximately constant, no matter the type of soil), and the resilient-response model for the granular material. When in-service pavements are considered, however, the last item remains approximately constant with variations in temperature and moisture, whereas the first two are highly susceptible to these environmental changes. Thus the asphalt-concrete surface-layer thickness, the granular base thickness, the modulus of elasticity of the asphalt concrete, and the resilient modulus of fine-grained soils at the break point constituted the bulk of the parameters varied in the ILLI-PAVE analyses. However, other inherent material properties of minor impact in the results were also varied.

Figure 1. Subgrade material models.



Researchers have attempted to relate flexible pavement performance to resilient-response parameters. Stresses, strains, and deflections were obtained from the ILLI-PAVE analyses. Response parameters found in the literature to be related to pavement performance were determined as follows:

1. Maximum radial strain at the bottom of the asphalt-concrete surface layer (eac),
2. Maximum surface deflection (d),
3. Maximum subgrade deflection (ds),
4. Maximum subgrade normal strain (ez),
5. Maximum base shearing stress (T),
6. Maximum subgrade normal stress (Sz),
7. Maximum subgrade shearing stress (Ts), and
8. Maximum subgrade deviator stress (Sdev).

A complete summary of the ILLI-PAVE analyses is presented in Figueroa (8).

In order to express stresses, strains, and deflections (response parameters) in terms of the four independent input variables (thicknesses and moduli), multiple regression analyses were performed. The ordinary least-squares estimation

Table 1. Algorithms developed from ILLI-PAVE results.

	Unstandardized Regression Coefficients for Independent Variables								Standardized Regression Coefficients for Independent Variables			
Parameter	o	p	q	r	s	R^2	$S\bar{x}$	F	tac	tbase	Eac	Eri
Group 1												
log(eac)	−3.33	0.028	−0.014	−0.000 13	−0.0051	0.600	0.0721	34.08[a]	0.188	−0.374	−0.619	−0.202
log(d)	−0.92	−0.079	−0.017	−0.000 14	−0.0248	0.905	0.0588	253.39[b]	−0.444	−0.278	−0.415	−0.583
log(ds)	−0.85	−0.088	−0.034	−0.000 14	−0.0313	0.949	0.0523	494.77[b]	−0.411	−0.451	−0.343	−0.608
log(ez)	−1.91	−0.123	−0.048	−0.000 20	−0.0288	0.935	0.0738	386.21[b]	−0.454	−0.509	−0.398	−0.445
log(T)	1.40	−0.108	0.014	−0.000 18	0.0045	0.766	0.1010	87.77[b]	−0.553	0.206	−0.488	0.096
log(Sz)	1.38	−0.078	−0.030	−0.000 12	0.0238	0.941	0.0468	426.99[b]	−0.432	−0.483	−0.347	0.554
log(Ts)	0.78	−0.067	−0.026	−0.000 12	0.0344	0.904	0.0673	251.77[b]	−0.330	−0.373	−0.300	0.709
log(Sdev)	1.08	−0.067	−0.026	−0.000 12	0.0343	0.904	0.0673	251.20[b]	−0.330	−0.373	−0.302	0.708
Group 2												
log Eac												
Eac = 690 MPa	−3.59	0.124	−0.012	–	−0.0006	0.896	0.0365	80.58[c]	0.879	−0.351	–	−0.026
Eac = 3450 MPa	−3.39	0.034	−0.017	–	−0.0074	0.831	0.0315	45.98[c]	0.354	−0.708	–	−0.453
Eac = 9650 MPa	−3.28	−0.075	−0.012	–	−0.0073	0.902	0.0262	85.66[c]	−0.718	−0.467	–	−0.411
Group 3												
log(d)	−0.73	–	−0.037	–	−0.0343	0.923	0.0603	77.94[d]	–	−0.578	–	−0.767
log(ds)	−0.65	–	−0.054	–	−0.0405	0.969	0.0476	205.53[d]	–	−0.666	–	−0.725
log(ez)	−1.73	–	−0.069	–	−0.0381	0.978	0.0445	290.74[d]	–	−0.773	–	−0.616
log(T)	1.64	–	−0.017	–	−0.0090	0.636	0.0551	11.36[d]	–	−0.639	–	−0.478
log(Sz)	1.47	–	−0.046	–	0.0267	0.963	0.0398	167.41[d]	–	−0.750	–	0.632
log(Ts)	0.75	–	−0.034	–	0.0438	0.908	0.0771	63.88[d]	–	−0.451	–	0.839
log(Sdev)	1.05	–	−0.034	–	0.0436	0.908	0.0769	63.92[d]	–	−0.451	–	0.839

Note: 1 MPa = 0.145 ksi.

[a] F = 4, 91; significant at α = 0.01. [b] F = 4, 107; significant at α = 0.01. [c] F = 3, 28; significant at α = 0.01. [d] F = 2, 13; significant at α = 0.01.

Figure 2. Nomograph based on the surface deflection.

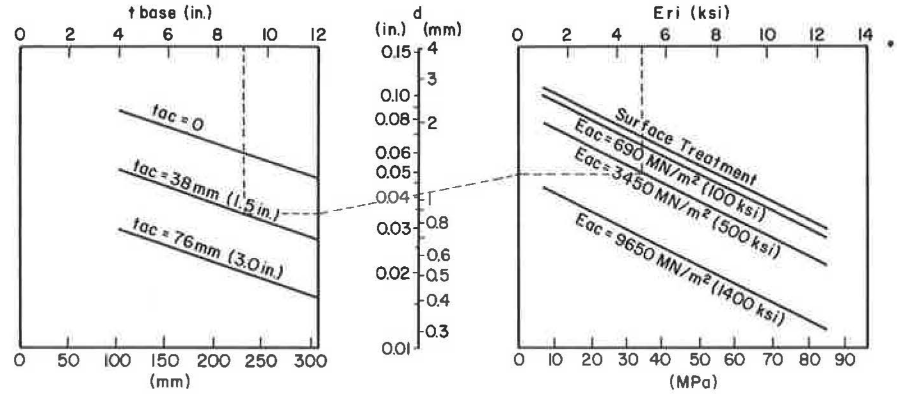


Figure 3. Nomograph based on the subgrade normal stress.

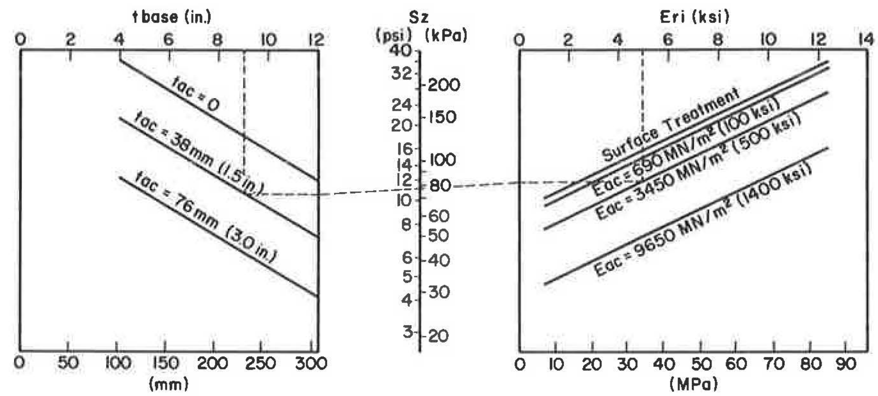


Figure 4. Nomograph based on the radial strain at the bottom of the asphalt-concrete layer [Eac = 3450 MPa (500 ksi)].

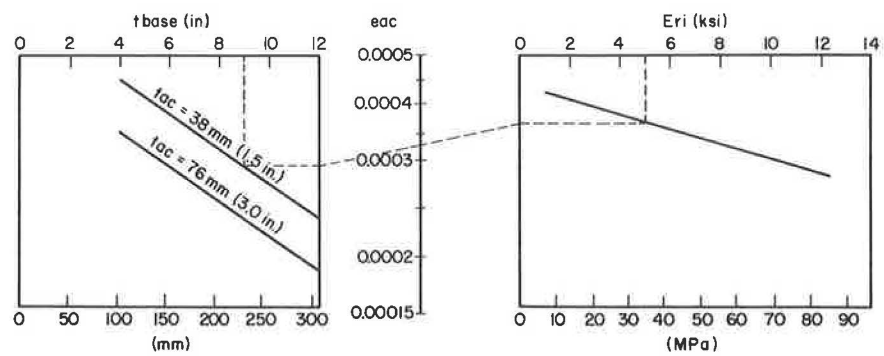


Figure 5. Nomograph based on the surface deflection (pavement sections in which tac = 0).

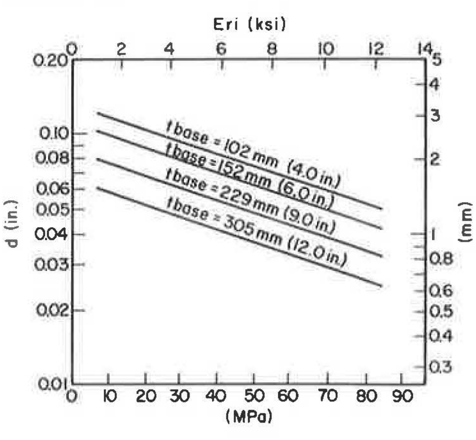
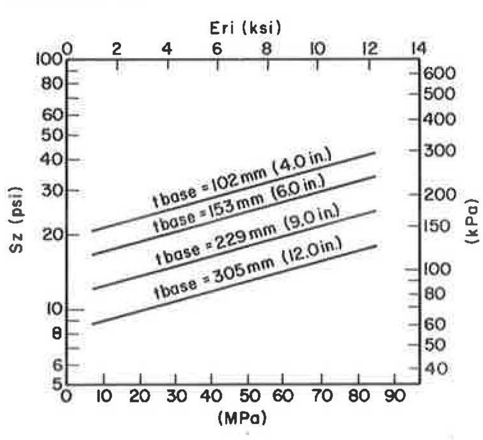


Figure 6. Nomograph based on the subgrade normal stress (pavement sections in which tac = 0).



technique was used. The general form of the predictive equation can be expressed as

$$\text{Log(response parameters)} = o + p \times \text{tac} + q \times \text{tbase} + r \times \text{Eac} + s \times \text{Eri} \quad (2)$$

where o = intercept or independent term and p,q,r,s = regression coefficients. After other possible forms of predictive equations were examined, the above form was confirmed to give the best correlation coefficients and the least standard errors of estimate. In addition, a stepwise multiple-regression analysis also indicated that the predictive equation including all four independent variables gave the best correlation coefficients and the least standard errors of estimate.

REGRESSION EQUATIONS

Selected regression equations are summarized in Table 1. These equations are valid only if the input parameters are given in standard units of thicknesses in inches and moduli values in pounds per square inch. Thus the calculated response parameters will be given in inches for deflections and in pounds per square inch for stresses. The coefficient of determination (R^2), the standard error of estimate (Sx), the variance ratio (F), and standardized regression coefficients are also indicated in the same table. In most cases, high R^2-values (above 0.9), high F-values, and low standard errors of estimate were obtained. It should be noted that the indicated Sx corresponds to the logarithm of the response parameter (dependent variable).

Three groups of regression equations are shown in Table 1. The equations of group 1 were determined by using the total 112 records existing for each response parameter, with the exception of the equation corresponding to the radial strain at the bottom of the asphalt-concrete layer (eac) when the sample size was equal to 96.

The significance of the predictive equation for eac is improved when the available data to develop the regression equations are analyzed in groups of constant modulus of elasticity of the asphalt concrete (Eac). Group 2 of Table 1 shows three different equations to predict eac, and they are applicable for the indicated values of Eac. The sample size was 32 in each case. Similar attempts to improve the prediction of the maximum-base shearing stress were unsuccessful.

Although pavement sections with tac = 0 were also included in the development of the regression equations of group 1, a separate regression analysis was performed on data obtained from pavement sections without an asphalt-concrete surface layer. In such cases, tac and Eac were eliminated from the regression equations (group 3 of Table 1). Thus, the sample size consisted of 16 points. It was expected that a more-realistic prediction of the response parameters to loading of pavements composed of a granular layer (crushed stone) placed on top of a generally fine-grained subgrade could be obtained.

When the overall variability of input strength parameters is considered, these regression equations appear adequate and simple to use for general pavement analysis within the range of assumed input parameters. As mentioned before, standardized regression coefficients are also specified in Table 1 for each regression equation. The absolute value of each one of these standardized regression coefficients indicates the importance of the input variable. The higher the absolute value of the standardized regression coefficient, the greater is the significance of the input variable. The importance of the subgrade support value, the resilient modulus of the subgrade at the break point (Eri), should be noted. In most of the predictive equations, the absolute standardized regression coefficient corresponding to Eri is the highest. This is an indication of the greater importance, in most cases, of subgrade strength (Eri) in determining the response parameters. However, when eac is to be predicted, the modulus of elasticity of the asphalt concrete becomes the input factor with the most influence, whereas Eri is the input factor with the least influence. Thus, Eac should be examined very closely when fatigue failure is considered.

When the regression equations of group 2 of Table 1 are analyzed, the asphalt-concrete surface-layer thickness seems to have the most influence. This observation is valid for the group of data that has the lowest and the highest modulus of elasticity of the asphalt concrete but not for the intermediate group, where the base thickness becomes the independent variable that has the most influence to determine eac.

If pavement sections containing a surface treatment and a granular base are considered (group 3 of Table 1), it is important to note that the subgrade normal stress is more efficiently changed with a variation in tbase than in Eri.

The nomographs presented in Figures 2 to 6 were developed based on selected algorithms taken from Table 1 for pavement sections containing a crushed-stone base. The response parameters obtained from these nomographs correspond to an applied wheel load equal to 40 kN (9 kips). The nomographs offer enough versatility to be expanded for the purpose of assembling a mechanistic design procedure. For example, the scale representing the response parameter could very well be replaced by the number of load repetitions (80-kN single-axle loads), providing suitable transfer functions of the form

$$\text{Response parameter} = f\,(\text{number of load repetitions}) \quad (3)$$

can be established. Also, the nomographs could be complemented with another plot representing the relationship for a given type of soil between the resilient modulus at the break point Eri and parameters such as degree of saturation and soil suction. In Figures 2 to 6, material properties and pavement-response-limiting criteria are commonly the input parameters from which the layer thicknesses can be determined. Similar nomographs can be developed for the remaining algorithms shown in Table 1.

CONCLUSIONS

Simple algorithms were developed to predict the resilient response of flexible pavements in terms of material properties and layer thicknesses. High coefficients of determination and low standard deviations indicated the reliability of these algorithms. Considering the overall variability of input values, these algorithms are adequate for general design. The algorithms are valid within the range of thicknesses and moduli used in their development.

A design process that includes the algorithms that predict the resilient response of flexible pavements (stress, strains, or deflections) in terms of material properties and layer thicknesses will be greatly simplified. These algorithms provide a fast, reliable, and economical estimate of the resilient response of a flexible pavement system subjected to traffic loading.

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Development of a Rationally Based AASHO Road Test Algorithm

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A new design algorithm for flexible pavements that uses performance data from the AASHO Road Test and layered-elastic theory is described. The algorithm, developed by correlating road test data with subgrade vertical strain calculated by the layered-elastic program ELSYM5, allows the characterization of seasonal variation in pavement strength and traffic volume and is implemented in a pavement design and management system. A comparison between the original design equation and the subgrade strain algorithm indicates that the latter is about 5.6 percent more accurate in correlating with road test performance data. However, the major advantage of the subgrade strain algorithm is that the pavement is characterized by using the modulus of elasticity of each layer, which allows consideration of seasonal variation and is a more rational basis for the extrapolation of results. When implemented in a pavement management system, the algorithm considers different axle loads separately by using the concept of cumulative damage; thus there is no need to convert mixed traffic to equivalent axle loads. It was also found that equivalent axle loads can vary significantly with different pavement structures.

Today in the United States many flexible pavements are designed or evaluated by using the procedure developed from the AASHO Road Test, i.e., the American Association of State Highway and Transportation Officials (AASHTO) Pavement Design Guides (1). This method and many other design methods are fundamentally empirical and therefore restrict the conditions under which the methods may validly be used. Some of the most difficult restrictions that engineers have had to overcome are conversion of mixed traffic into a single unit, such as 80-kN (18-kip) equivalent single-axle loads; consideration of seasonal variation in pavement strength; adjustment for different regional climatic conditions; and characterization of the relative strengths of different pavement materials.

In an effort to eliminate some of these restrictions, the Transportation Group at the University of Texas at Austin has developed a new design algorithm for flexible pavements. The algorithm is based on the relationship between compressive strain at the top of the subgrade and number of repetitions to a terminal level of serviceability derived from the AASHO Road Test data. This development, sponsored through a cooperative agreement with the U.S. Forest Service, was carried out as part of a project to improve an existing pavement design and management system (PDMS) also developed at the University of Texas (2). The design problems mentioned above are critical to U.S. Forest Service engineers, who design and manage a road network of more than 320 000 km (200 000 miles). Of particular importance to these engineers is the consideration of seasonal variation in pavement strength and in traffic volume. Forest roads, and even many state and county roads in northern climates, often have axle-load restrictions during spring thaw periods because of the weakened condition of the pavement. The improved pavement management system will make it easier for engineers and planners to evaluate the economic trade-offs involved in these spring restrictions.

This paper explains how the new design algorithm was developed and compares results with the AASHTO design method. Applications are discussed, particularly those that consider seasonal variation. Conclusions and an outline for further research are summarized at the end of the paper.

AASHTO DESIGN METHOD

The AASHTO pavement design method was developed by using the results from the AASHO Road Test conducted October 1958 through November 1959 near Ottawa, Illinois. This carefully engineered experiment included six loops and 468 test sections of asphalt pavement that were subject to traffic loads ranging from 9-kN (2-kip) single axles to 214-kN (48-kip) tandem axles. These test sections were monitored to determine how different pavement thicknesses and traffic loads affected pavement performance. Performance was subjectively measured by a panel of raters by using a Present Serviceability Rating that ranges from 0 for very poor to 5 for excellent. Correlation of the panel ratings of performance with measurements of cracking, rut depth, and roughness gave a more quantitative Present Serviceability Index (PSI), so that the condition of the pavement could be accurately determined by measurement, rather than by assembling a panel of raters.

The analysis of road-test data resulted in a design method that determines the strength required of a pavement structure to withstand a given number of load applications before the performance of the pavement reaches a given minimum (or terminal) PSI. The required strength of a pavement is given in terms of the structural number (SN), which is the thickness of each pavement layer, multiplied by a strength coefficient for each layer, summed over all the layers of the pavement, as in Equation 1:

$$SN = a_1 d_1 + a_2 d_2 + \ldots a_n d_n \tag{1}$$

where

a_n = strength coefficient for layer n,
d_n = thickness of layer n (in), and
n = number of layers above subgrade.

The AASHTO strength coefficients (a_i) for each layer of material at the road test were determined empirically (3) by using the different layer thicknesses as independent variables to predict the SN. The coefficients from the regression equation were then related as the material strength coefficients for the road test. A problem developed when different material strength coefficients were determined for different pavement thicknesses (see Table 1). The final values determined for materials at the road test were taken as the average of the coefficients from each loop.

For a given pavement SN and given minimum PSI, the AASHTO design equation computes the number of allowable applications of a certain load, as in Equation 2 (1):

$$\log W_{t_x} = 5.93 + 9.36 \log (SN + 1) - 4.79 \log (L_x + L_2) + 4.33 \log L_2 + G_t/\beta_x \tag{2}$$

where

W_{tx} = number of applications of axle load x,
L_x = magnitude of axle load (kips),
L_2 = axle type (1 for single and 2 for tandem),
G_t = $\log [(4.2 - p_t)/(4.2 - 1.5)]$,
β_x = $0.40 + [0.081(L_x + L_2)^{3.23}/(SN + 1)^{5.19} L_2^{3.23}]$, and
P_t = terminal value of PSI (value of PSI used to indicate failure).

Equation 2 only serves to predict pavement life for the same conditions found at the AASHO Road Test and does not consider regional differences of climate or subgrade soil types. In addition, to design for mixed traffic it is necessary to convert various axle loads to single-equivalent axle loads.

These factors are considered in the AASHTO design equation most familiar to pavement engineers, shown here as Equation 3:

$$\log W_{t_{18}} = 9.36 \log (SN + 1) - 0.20 + \left(G_t/\left\{0.40 + [1094/(SN + 1)^{5.19}]\right\}\right) + \log (1/R) + 0.372 (S_i - 3.0) \tag{3}$$

where

W_{t18} = number of 80-kN (18-kip) axle applications,
R = regional factor (ranging from about 0.5 to 5.0, where the higher number represents a more severe climate), and
S_i = soil support, which normally ranges from 1.0 to 10.0 and is correlated with different soil strength tests, such as California bearing ratio (CBR).

The R and S_i terms are qualitative and were inserted to permit the regional factor and subgrade soil type to be reflected in design, as would be rationally expected. The S_i term has no rational basis and was selected as a compromise between the many potential test methods in use by various agencies in the United States at the time of the road test analysis. The S_i values from the AASHO Road Test were arbitrarily set at 3.0 and 10.0 for the subgrade soil and the crushed-stone base, respectively. Thus, by using these two control points each agency was permitted to develop a correlation between the S_i scale and the agency's test methods.

The R term is used for climatic variation and was inserted to permit adjustments for conditions other than those at the road test. It also has no rational basis, and each agency is left to determine the proper adjustment factor(s) for its area(s). In essence, the term is simply a divisor for the load repetitions from the AASHO equation. A pavement structure lasting x applications at the road-test site would only last x/3 applications for an R of 3.0.

Except for a few states, the AASHTO design method is used by most highway departments in the United States to design or evaluate designs of flexible pavements (4). To determine how accurately the AASHTO design equation models the performance observed at the AASHO Road Test, actual values of SN, axle load, and terminal PSI at the road test were used as inputs to Equation 2. The calculated number of applications from the AASHTO equation was then plotted against the actual observed number of weighted applications for a given test section at the road test. [The weighted applications are the same data used to develop the AASHTO equation and represent original observations corrected for seasonal variations (3)]. This involved plotting 523 observations, which included all the results from the main block of road-test experiments (3). Figure 1 contains this plot, and indicates graphically how accurately the AASHTO equation predicts the AASHO data. Statistically, the coefficient of determination (R^2) is 0.738, indicating that 73.8 percent of the variation in the AASHO data is described by the AASHTO equation, and the standard error of estimate (S_e) is 0.300. Examination of Figure 1 reveals that the widest range of scatter about the line of equality corresponds to an AASHO data value of about log 6.0. This results from the fact that at the end of the road test, after a total of 1 114 000 applications, some pavements remained that had not reached the lower performance level of the terminal PSI. These pavements that have higher PSI values are not predicted as well as when a lower terminal PSI level is used.

Background information just given on the AASHTO design method indicates some of the problems encountered with the method, particularly in extrapolating beyond the conditions at the road test. Variables such as R, S_i, and material strength coefficient are used to design for different climatic and material conditions, but the basis for their use is very qualitative and subject to speculation. It was felt that some other algorithms that would have a more rational basis could be developed from the AASHO Road Test data; these algorithms could then be more realistically extrapolated to other conditions.

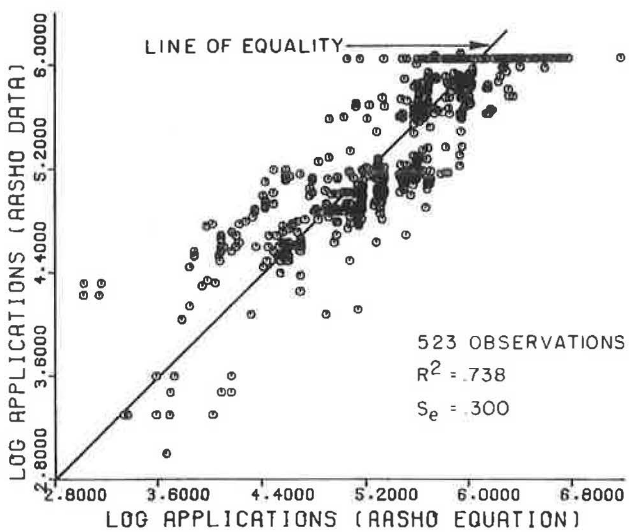
DEVELOPMENT OF THE NEW DESIGN ALGORITHM

The concept in developing the new design algorithm was that some stress-strain parameter of the pave-

Table 1. AASHO Road Test material strength coefficients.

Material	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6	Weighted Average
Asphalt (a_1)	0.83	0.44	0.44	0.47	0.33	0.44
Base layer (a_2)	0.25	0.16	0.14	0.14	0.11	0.14
Subbase layer (a_3)	0.09	0.11	0.11	0.11	0.11	0.11

Figure 1. AASHTO equation as predictor of AASHO data.



ment could be incorporated into a design equation similar to the form of the AASHTO design method. In this way, the procedure would have a more mechanistic orientation and therefore be more adaptable to conditions outside the range of the road-test data. To determine the validity of this concept, stress-strain characteristics of different pavements under various loads were calculated by using the layered-elastic theory program ELSYM5. This program is a modification of the Chevron layered-elastic program developed at the University of California at Berkeley (5).

Input data required for the program include the thickness, Poisson's ratio, and modulus of elasticity of each layer in the pavement; the tire pressure; and the magnitude of the applied wheel loads. The ELSYM5 program was used to analyze the same pavements and axle loads used at the road test in order to allow a comparative analysis with the AASHO Road Test performance data. The moduli of elasticity used were those determined from laboratory tests on the materials used at the road test and are listed below (4). Values of Poisson's ratios, found to have a very small effect on the ELSYM5 program, were estimated (1 MPa = 145 psi):

Material Property	Value
Modulus of asphalt concrete (MPa)	3103
Modulus of base layer (MPa)	207
Modulus of subbase layer (MPa)	103
Modulus of subgrade (MPa)	20
Poisson's ratio of asphalt concrete	0.30
Poisson's ratio of base layer	0.40
Poisson's ratio of subbase layer	0.40
Poisson's ratio of subgrade	0.45
Tire pressure (kPa)	520

The material properties shown above are normalized values that reflect a range in conditions at the AASHO Road Test. For example, the modulus for asphalt concrete would vary daily as well as seasonally. The moduli for the base, subbase, and subgrade layers would vary seasonally, depending on the moisture content and density in the layers.

ANALYSIS PROCEDURE

At the beginning of the analysis, a small group of observations was made to see whether parameters such as tensile strain at the bottom of the asphalt, stress in the subgrade, and compressive strain at the top of the subgrade would correlate with results from the AASHTO design equation. It was quickly apparent that the most promising parameter was the compressive strain at the top of the subgrade and that a further analysis should be completed. Others, after analyzing some selected data from the road test (6), have used the compressive subgrade strain as a prediction of performance or distress, particularly for rut depth. There was, however, no evidence to indicate that compressive subgrade strain had been examined as an indicator of performance for all test sections at the road test. Computations of subgrade strain by using ELSYM5 were made on all combinations of road-test pavements and axle loads. A regression analysis that used the stepwise program STEP01 was then made to see how well the subgrade strain parameter as an independent variable could predict the AASHO Road Test performance data. However, because of the wide scatter found with the AASHTO equation for observations where there were high values of PSI, it was decided to transform the road-test data in the following way: log W_{tx} was transformed to $\log\{W_{tx} \div \sqrt{[(4.2 - p_t)/(4.2 - 1.5)]}\}$, where all terms are as defined in Equation 2. In this way, the observations that have a high value of p_t are transformed to the number of applications required to reach a p_t of 1.5. The square-root function allows for a nonlinear PSI curve from construction to failure, with the deterioration of the pavement accelerating more as its condition becomes worse.

The regression equation computed from the STEP01 program was the following:

$$\log\left\{W_{tx}/[(4.2 - p_t)/(4.2 - 1.5)]^{1/2}\right\} = 2.151\,22 - 597.662\,\epsilon_{SG} - 1.329\,67\,(\log \epsilon_{SG}) \tag{4}$$

where ϵ_{SG} = compressive vertical strain at top of subgrade as calculated from ELSYM5. Statistically, this equation has an R^2 of 0.810 and an S_e of 0.240. This is significantly better than the prediction of the AASHTO equation, as is illustrated in Figure 2.

Equation 4 predicts the anticipated number of applications to reach a p_t of 1.5. However, for designs requiring a different terminal PSI, the equation should be put in the same form as the AASHTO equation, as shown in Equation 5:

$$\log W_{tx} = 2.151\,22 - 597.662\,(\epsilon_{SG}) - 1.329\,67\,(\log \epsilon_{SG}) + \log\left\{[(4.2 - p_t)/(4.2 - 1.5)]^{1/2}\right\} \tag{5}$$

Figure 3 shows how well Equation 5 predicts the same 523 road-test observations predicted by the AASHTO equation in Figure 1. A comparison of Figures 1 and 3 reveals that the strain equation has less scatter about the line of equality, particularly for the observations at the end of the road test. Statistically, Equation 5 has an R^2 of 0.794, meaning that it predicted the variation of road-test data approximately 5.6 percent better than did the AASHTO equation. The S_e of Equation 5 is

0.266, an improvement over the value of 0.300 for the AASHTO equation.

Equation 5 represents the best-fit equation for the average material properties presented in the text table above. An improved statistical fit might be obtained by inputting the material properties to represent the seasonal variation during the road test. It is beyond the scope of this paper to consider these seasonal variations in road-test material properties, but future developments described below will consider these variations.

It is also recognized that Equation 5 is, in essence, a performance equation for one subgrade soil type. The coefficients may quite possibly vary with different soil types, but only extensive investigations could ascertain this. The equation considers different soil types by changing the modulus of elasticity of the subgrade.

The real benefit of the subgrade strain design equation (Equation 5) is not the fact that it is more accurate than the AASHTO equation but that it represents a more rational and mechanistic characterization of the pavement parameters. The subgrade strain is computed by using the modulus of elasticity of the pavement layers and subgrade, rather than empirical material strength coefficients and S_i. The modulus of elasticity is a far more universal and understandable parameter and is measurable in the laboratory through resilient-modulus testing techniques. With the strain design equation, there is no need for a term like R because the modulus of elasticity can be used to characterize the environmental effects of climate on materials, as is discussed in the implementation section of this paper.

Figure 2. Strain equation as predictor of transformed AASHO data.

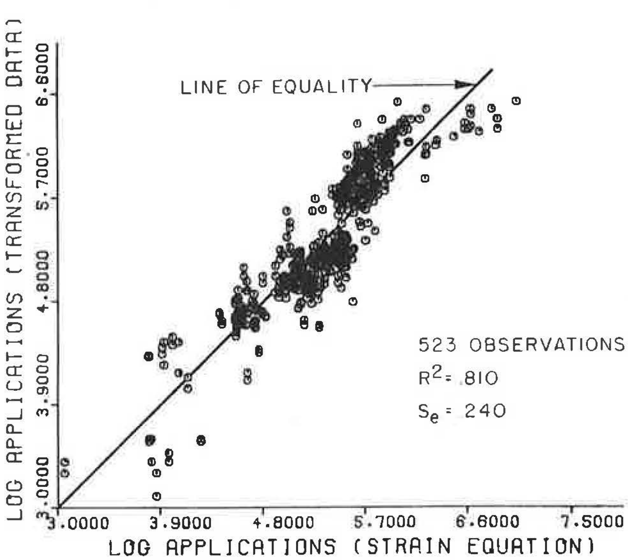
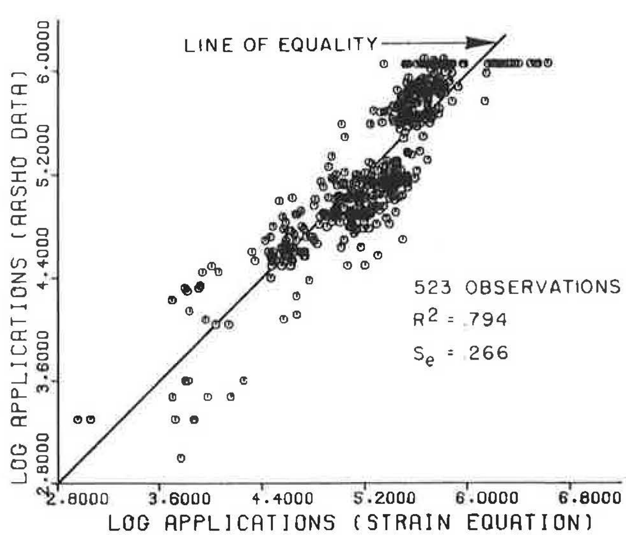


Figure 3. Strain equation as predictor of AASHO data.

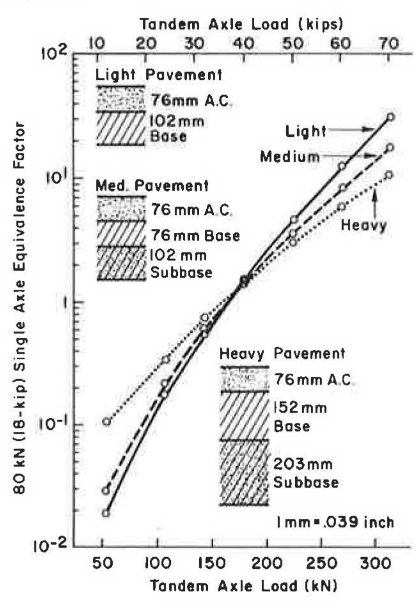


LOAD EQUIVALENCY FACTORS

Use of the strain equation eliminates the need to convert mixed traffic into equivalent 80-kN (18-kip) axle loads. The effect of various axle loadings can be analyzed separately, and this is discussed later in the paper. In an exercise to again compare results of the AASHTO design method with those of the subgrade strain equation, 80-kN axle equivalencies for a number of cases were calculated by using the strain equation and were then compared with the AASHTO equivalencies. It was found that the 80-kN equivalencies from the strain equation will vary substantially with the pavement structure, whereas AASHTO equivalencies vary only slightly with a change in SN. This is illustrated in Figure 4, where 80-kN equivalencies for the strain equation are plotted versus tandem-axle load for three different pavement structures. Figure 5 then shows the same equivalencies from Figure 4 for the light and heavy pavement structures compared with the AASHTO equivalencies. There is only one line for the AASHTO equivalencies in Figure 5 because with the logarithmic scale virtually no difference exists between the AASHTO equivalency lines for different pavement structures.

The suggested hypothesis for the difference in equivalency factors with the strain equation comes from the consideration of the stress sensitivity of the pavement materials. It has been found that with a larger confining pressure, or a smaller deviator stress, the resilient modulus of a fine-grained material will increase (7). This factor is not reflected in a layered-elastic program such as ELSYM5,

Figure 4. Plot of 80-kN single-axle equivalencies computed from strain equation.



which does not modify the moduli of the materials as a function of confining pressure or deviator stress. On the other hand, because the AASHO Road Test performance data were subject to the stress sensitivities of the materials at the road test and the strain equation is a regression of that data, the equation should relate those same stress-strain characteristics. This hypothesis can be further evaluated by observing the equivalency relationships in Figure 4. Remembering that the axle equivalencies are computed by $e_x = W_{t18}/W_{tx}$ = 80-kN (18-kip) equivalence factor, the difference between equivalencies for high loads can be noted for the three pavements. As the load increases, the value of W_{tx} decreases, causing the value of the 80-kN equivalence to increase. For a heavy pavement, the confining pressure is more than that for a light pavement and the deviator stress is less, thereby resulting in a higher modulus of the pavement materials. This causes a larger value for W_{tx} and W_{t18}, but the effect on W_{tx} is greater because it is related to a heavier load. For this reason the ratio of W_{t18}/W_{tx} is larger for light pavement structures than for heavy ones, as is shown in Figure 4. The same concept is used when the loads become lighter and W_{tx} increases. From a light to heavy pavement, W_{tx} and W_{t18} both increase, but this time W_{t18} is related to the heavier load and increases more on a relative basis. This causes W_{t18}/W_{tx} to be smaller for the light-pavement structure than for the heavy one.

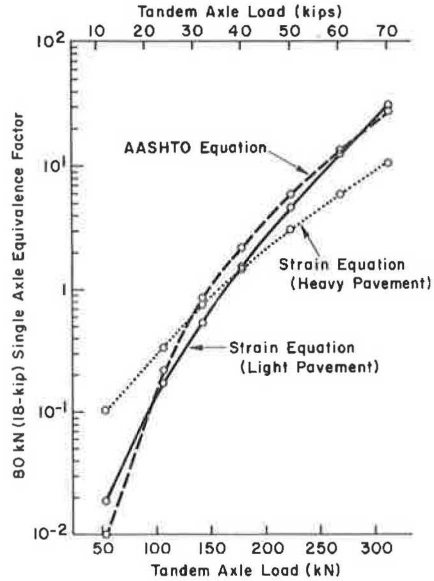
In general, the relationship shows that difference in load magnitude has more of an effect on weak pavements than on strong ones. It is encouraging to find that the strain equation is influenced by stress sensitivity, since it is not practical at this time to compute the strain by using a costly iterative process that would take this stress sensitivity into consideration. It will, however, be investigated in future development work, as discussed below.

We recognize that most agencies have developed traffic projections by using the load equivalencies determined from the AASHO Road Test. The new algorithm does not in any way limit the application of these projected equivalent 80-kN (18-kip) loads for a given facility. Basically, the new algorithm could be solved for an equivalent 80-kN single-axle load for a given pavement structure and all solutions could be made in terms of the standardized real load. The new algorithm can therefore be inserted as a design equation without interrupting the normal traffic projection procedures if any agency so desires.

IMPLEMENTATION OF SUBGRADE STRAIN EQUATION IN A PAVEMENT MANAGEMENT SYSTEM

As previously stated, the University of Texas developed PDMS for the U.S. Forest Service as an instrument for designing and maintaining flexible pavements, and it is for this system that the new design procedure was developed. By using the AASHTO design method alone, engineers were unable to characterize seasonal variation in pavement strength. R was a very crude means of considering seasonal variation, but there was no way to also consider any seasonal variation in traffic. The U.S. Forest Service, like many transportation authorities in northern climates, often restricts axle loads on roads during the spring thaw periods when most pavements are in a very tender condition. Design engineers needed some way to evaluate the economic difference between building a pavement strong enough to withstand the spring traffic and building a weaker pavement where loads are restricted during the worst times of the year. An example of such use of the new design procedure in PDMS is summarized here.

Figure 5. Comparison of equivalence factors computed from AASHTO and strain equations.



The conceptual approach of the subgrade strain design procedure is illustrated in Figure 6. For a single-axle load of 80 kN (18 kips) on a given pavement structure, there are W_{t18} applications of that load before failure, as computed by Equation 5. If the load is a 214-kN (48-kip) tandem axle for the same pavement, there are W_{t48} applications of that load before failure. When mixed traffic of both loads is applied to the pavement, then both contribute to pavement distress. If W_{t18} is equal to 800 000 applications and there are 200 000 80-kN applications (n_{18} = 200 000) at a time t, then at time t the cumulative damage is n_{18}/W_{t18} or one-fourth the damage at failure. If W_{t48} is equal to 300 000 applications and in the same time t there have been 225 000 214-kN applications (n_{48} = 225 000), then the total damage is as follows: $(n_{18}/W_{t18}) + (n_{48}/W_{t48}) = (1/4)+(3/4) = 1$. As soon as the cumulative damage sums to a value of 1, then the pavement is predicted to have failed. In this way each element of a mixed traffic load can be considered separately, and there is no need for axle-load equivalencies.

The same concept is used to consider seasonal variation in pavement strength, as shown in Figure 7. At a given time of the year for a certain pavement structure, the modulus of elasticity of each layer can be used to characterize the strength of the pavement materials. The moduli of pavement layers could change as a result of heavy rainfall, poor drainage, frozen conditions, dry weather, or almost any environmental effect. During the summer a certain load may produce a calculated strain ϵ_{sum} that corresponds to the number of load applications to failure of W_{t-sum}. During a spring thaw condition, the modulus of the subgrade may be very low, leading to a high strain ϵ_{spr} and low number of applications to failure W_{t-spr}. By considering the amount of traffic in the summer n_{sum} and spring n_{spr}, the cumulative sum of n/W_t can again be used to predict when the pavement will fail. Table 2 gives an example of combining seasonal variation of pavement strength and traffic [$\Sigma(n/W_t)$ = 0.200].

Figure 6. Strain algorithm results for 80-kN single-axle and 214-kN tandem-axle loads.

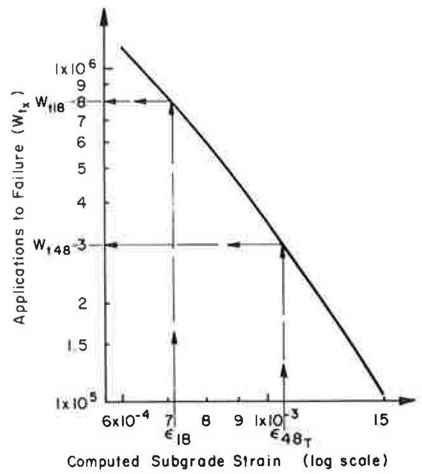


Figure 7. Strain algorithm results for seasonal variation.

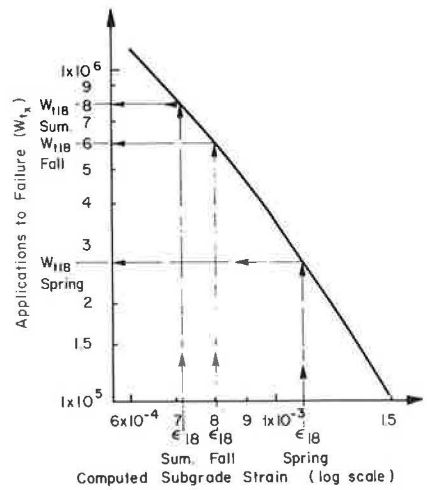


Table 2. Example of 80-kN single-axle and 214-kN tandem-axle loads during one year.

Season	$W_{t_{18}}$	$W_{t_{48}}$	n_{18}	n_{18}	n/W_t
Summer	800 000	300 000	10 000	12 000	0.053
Fall	600 000	225 000	7 000	8 000	0.047
Spring	250 000	100 000	5 000	8 000	0.100

The results from Table 2 would indicate that the pavement in question would be predicted to fail after approximately 5 years of the same traffic. By using PDMS, the user specifies the minimum number of years of service required before pavement rehabilitation is allowed. In the example given, if 10 years had been specified as the minimum time before rehabilitation and the pavement was estimated to last only 5 years, then the program would automatically increase the thickness of the pavement structure until the pavement lasted at least 10 years before failing $[\Sigma(n/W_t) = 1]$.

This new design procedure, therefore, eliminates the need for considering S_i or R, and instead the pavement is characterized by using the modulus of elasticity for each layer. Because the design method is used in a computer program, the number of axle weights or number of periods in the year may be divided into as many units as desired. It is very easy for the program to sum all the possible n/W_t values.

In this entire analysis, the values of subgrade strain were calculated by using the program ELSYM5. Because the PDMS program sometimes considers hundreds of candidate design strategies, it would be too costly to have ELSYM5 calculate the subgrade strain for each design. For this reason, a regression analysis of a statistically designed factorial of ELSYM5 solutions was made to see whether an equation could be derived that, given the same inputs, would predict accurately the results from ELSYM5. In a trial analysis of this type with two-layer pavements, it was found that the regression equation explained 99 percent of the variation in the ELSYM5 calculations and would therefore be acceptable to use in the PDMS program to calculate subgrade strain. More ELSYM5 regression analyses of this type will be made in the future to include more pavement layers and a wider range of input values.

CONCLUSIONS

Based on this study, the following specific conclusions may be derived.

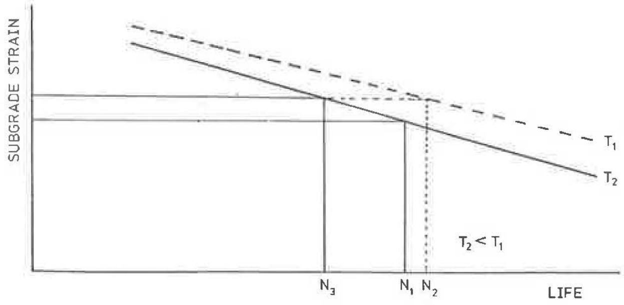
1. Equation 5 presented herein predicts the number of applications to reach a desired terminal serviceability based on a predicted subgrade strain value. The equation improved the statistical correlation of the AASHO data, improving the R^2 from 0.738 to 0.794 and the S_e from 0.300 to 0.266.
2. By using Equation 5, the designer may consider any regional or seasonal variation of material properties to whatever level of sophistication desired, i.e., from one set of conditions to any number of conditions desired.
3. Equation 5 is, in essence, a performance equation based on one subgrade soil type, but it may more reliably be extrapolated to other soil types since the extrapolation is on a rational basis.
4. The material properties of the surface, base, and subbase layers are more rationally characterized by using modulus of elasticity than by using empirical material strength coefficients. Thus, design values may be established by testing new materials rather than by relying on engineering judgment.
5. The computations may be made for each axle weight, thereby eliminating the need for determining equivalent axle loads.
6. The analysis shows that if axle equivalencies are evaluated they will vary significantly for different pavement structures, whereas AASHTO equivalencies show a small variation.
7. The algorithm presented in Equation 5 may be substituted for the AASHTO design equation used by many agencies with a minimal effect on the normal design techniques.

OUTLINE OF FUTURE DEVELOPMENT

Future development work on the algorithm presented in this paper will concentrate in three areas.

1. Stress sensitivity: A capability will be developed in the PDMS program to characterize the modulus of elasticity of granular or fine-grained materials as a function of the calculated in situ stress condition. This will increase the mechanistic viability of the algorithm and allow for more reliable extrapolation of results.
2. Seasonal variation of AASHO Road Test data:

Figure 8. Subgrade strain: life relationships at two temperatures.



The material used at the road test will be characterized seasonally, including the consideration of stress sensitivity mentioned above. This will eliminate the need to use normalized values of material properties and will more accurately model the seasonal variation in pavement performance.

3. Remaining life of pavement structure: Because pavements do not deteriorate linearly with time or traffic, a capability will be developed to express the rate of deterioration as a function of the present condition of the pavement structure.

The Forest Service of the U.S. Department of Agriculture is also initiating a new data base system that will collect and store information concerning typical pavement characteristics at different times of the year. This development will expand the data from which inputs for the PDMS program are selected and from which performance equations are derived.

The new design procedure presented in this paper is considered to be an excellent framework for the future that can be modified and improved as more information becomes available.

ACKNOWLEDGMENT

The research and development work discussed in this paper was conducted under a cooperative agreement between the University of Texas at Austin and the Forest Service of the U.S. Department of Agriculture. The purpose of the project is to review and improve the Flexible Pavement Design chapter of the Forest Service Transportation Engineering Handbook.

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Discussion

A.F. Stock

In their paper, Luhr and McCullough use the subgrade strain criterion for a cumulative damage analysis of pavement systems on the basis of a linear summation of cycle ratios (Miner's hypothesis). Since the subgrade strain criterion was developed from back analysis of pavements for average conditions, it is incorrect to use it in this way; therefore, conclusions based on this type of analysis are highly suspect.

Consider, first of all, the derivation of the subgrade strain criterion. Analysis carried out by following the AASHO Road Test indicated that the development of permanent deformation in the flexible pavement test sections could be related to the strain on the subgrade calculated by an elastic analysis. This analysis was carried out for average conditions, particularly with regard to temperature, and provides a relationship of the type shown by the solid line in Figure 8.

Asphalt stiffness decreases as temperature increases. If a pavement is analyzed for several temperature conditions, under constant loading conditions the strain on the top of the subgrade will increase with the temperature. If the unique subgrade strain criterion is used, such an analysis predicts that pavement life will decrease (from N_1 to N_3) as the temperature increases, which is quite logical. However, if a relationship between performance and subgrade strain for the same pavement is derived at a higher temperature, the line so derived will be as shown by the dotted line in Figure 8, i.e., above the original line. Use of this second line, which is relevant to the higher temperature condition, indicates that for this condition the pavement may have an increased life (N_2). This is not logical and indicates clearly the major limitation of the subgrade strain criterion, i.e., that, since it was derived for a given set of conditions, it cannot logically be applied to any others.

A similar argument follows for different axle loads.

It is not the purpose of this discussion to suggest that the subgrade strain criterion is of no value in estimating the life of a pavement structure. It is a simple indicator of performance and, provided that this is borne in mind, it is a useful tool. However, its simplicity and the background of its derivation make it inappropriate to use in a sophisticated cumulative-damage calculation.

Authors' Closure

Stock's discussion makes a valid point concerning the use of only one modulus of elasticity to characterize the asphalt-concrete layer; however, the discussion includes some misconceptions about the algorithm that we would like to clarify.

As was stated in the paper, the stiffness of asphalt concrete in a pavement structure can vary seasonally as well as hourly. The concept of using one normalized modulus of elasticity to characterize an asphalt concrete with varying stiffness has been described by Shahin and McCullough (8). We feel that the use of a normalized asphalt-concrete stiffness does not seriously detract from the usefulness of the algorithm.

It is not always true that pavement life decreases with increased temperature, as is stated in the discussion. There is a probability that increased rutting will occur at the lower stiffness of asphalt concrete associated with higher temperatures, but the reduced stiffness will also reduce the amount of temperature and fatigue cracking. In addition, the stiffness of base, and especially subgrade, materials commonly increases during seasons associated with higher temperatures. This additional stiffness would act to decrease, rather than increase, the subgrade strain. It is imperative to remember that the subgrade strain algorithm is a predictor of PSI, which is primarily associated with roughness and not singularly a predictor of rutting or cracking.

We feel that the effect of temperature and, therefore, the effect of asphalt-concrete stiffness on subgrade strain as indicated in Figure 8 is somewhat exaggerated. If relationships were derived for different temperatures, they would lie much closer to the solid line than to the dashed line associated with T_1. Actually, this is one of the benefits associated with an algorithm based on subgrade strain. The algorithm is not dominated by asphalt stiffness but, rather, is affected by the combination of thickness and material properties of all the layers in the pavement structure.

Stock states in the discussion that an argument similar to that proposed concerning the asphalt modulis is also applicable for different axle loads. This is not the case, since 10 different axle loads ranging from 9-kN (2-kip) single-axle to 214-kN (48-kip) tandem-axle loads were included in the subgrade strain analysis.

We appreciate Stock's comments. As we stated in the paper, development work is continuing in order to consider a more complex seasonal characterization of road-test data.

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Interactive Pavement Behavior Modeling: A Clue to the Distress-Performance Problem

W. KIRK SMEATON, S. S. SENGUPTA, AND RALPH HAAS

A framework and suggested methodology for identifying objective relationships between pavement distress and performance is presented. The nature of the problem and the suitability of two widely used models of pavement performance are reviewed. The status of pavement behavior modeling is assessed, and a more comprehensive theory is advanced based on the hypothesis that (a) the levels of different types of pavement distress behavior are interdependent through time and (b) pavement behavior elements are recursive in that they depend on their own historical values. The viability of the theory is investigated through postulating a deterioration mechanism for flexible AASHO Road Test sections. A preliminary interactive model involving three behavior subsystems (rutting, cracking, and roughness) is described.

In recent years, pavement engineers have become increasingly aware of the need to make more efficient use of pavements in service. The very noticeable shift in pavement expenditures from capital construction to the maintenance and repair of existing pavements bears testimony to this need.

The efficient use of existing pavements involves the programming of expenditures over large pavement networks so as to maximize the total benefits. These benefits are directly related to pavement performance, mainly from the user's perspective.

However, because pavement performance is affected by distress, because it is distress that is treated in maintenance, and because such maintenance varies with the type and degree of distress, it would be desirable to take the appropriate improvement action at the best time(s) in order to have the maximum impact on performance. To achieve this it is first necessary to know the relationships between distress and performance, as schematically illustrated in Figure 1.

The need for relating pavement distress to performance was identified as the primary research need in a workshop of top-ranking pavement experts in 1970 (1). It was subsequently considered for several years by a Transportation Research Board (TRB) task force and discussed in various forums, including a two-day workshop at the 1977 TRB conference. This task force also conducted a survey of American state and Canadian provincial highway agencies to review the current practices in this area (2). It was found that, although some agencies have attempted to relate distress to performance, all such investigations are in very preliminary stages.

This paper considers the nature of relationships between pavement distress and performance in the context of an interactive approach to modeling pavement behavior. Specifically, the objectives are to

1. Present an interactive approach to modeling pavement behavior,
2. Present and discuss a preliminary investigation of the workability of this modeling approach to the development of pavement distress-performance relationships by using AASHO Road Test data, and
3. Identify and discuss areas of refinement and deficiency that are requisite to the development of

Figure 1. Schematic illustration of the problem of relating pavement distress to performance.

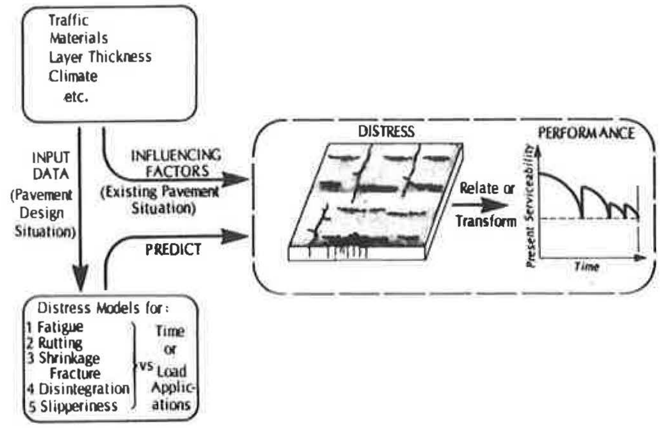
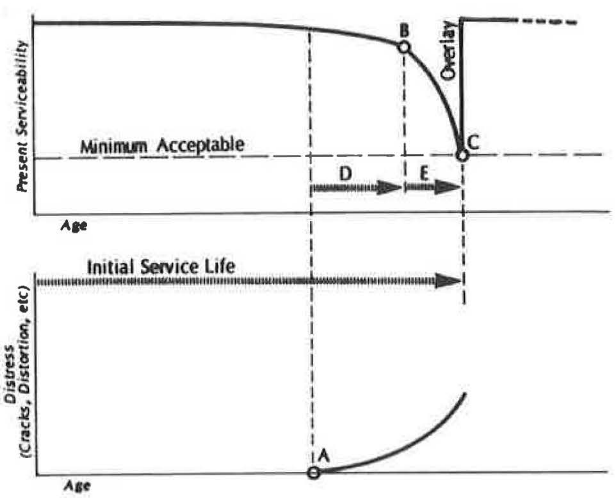


Figure 2. Time-delay nature of distress-performance relationships.



NOTES:

1 A represents the age at which distress first appears.

2 B represents the age at which distress begins to affect performance.

3 C represents the age at which pavement has to be overlaid, reconstructed, or continue to be maintained at a minimum acceptable serviceability level.

4 D represents the time delay between the appearance of distress (A) and the age at which distress begins to affect performance (B).

5 E represents the period of accelerated loss of serviceability attributable to the appearance and spread of distress.

objective relationships between pavement distress and performance.

INTERACTIVE NATURE OF PAVEMENT BEHAVIOR

Elements of Pavement Behavior

Historically, pavement behavior has been characterized in terms of performance and distress. Performance is the most significant operating characteristic of a pavement. It is usually defined as the variation of the level of service (serviceability) provided to pavement users with time and/or traffic. The two most widely used measures of pavement serviceability in North America are the Present Serviceability Index (PSI) in the United States and the Riding Comfort Index (RCI) in Canada. Both concepts relate objectively measurable pavement behavior to the average user's opinion of serviceability. The main consideration in the user's conception of performance is riding comfort, which is usually estimated by measured roughness.

Pavement distress, in general, is a form of limiting pavement behavior characterized by perceptible evidence of physical deterioration of the pavement. The three major forms or types of distress in North American flexible pavements are permanent deformation (primarily rutting), low-temperature shrinkage fracture, and fatigue cracking.

Pavement distress mechanisms are related to the physical or mechanical behavior of the pavement and its various components. Pavement engineers have hypothesized distress mechanisms for almost every form of distress and, accordingly, have developed models that can estimate the occurrence of specific forms of distress with reasonable accuracy.

The role of distress-prediction models has been primarily to check pavement designs against acceptable levels of distress (3). As a result, many of the models developed to date are based on failure criteria rather than on providing estimates of the amounts of distress to be expected. In order to develop objective relationships between pavement distress and performance, it is desirable to develop models that can estimate the amount of distress.

Probably the first attempt to relate pavement performance to objectively measurable distress was the PSI concept developed for the AASHO Road Test. The relationship developed estimated subjective user opinion of present serviceability from present amounts of roughness, cracking, patching, and rutting. The use of present amounts of distress in this formulation is the major drawback of the PSI concept (4). Because there is a time lag between the appearance of distress and the consequent loss in serviceability, as shown schematically in Figure 2, the PSI concept does not adequately relate pavement distress to performance. The varying nature and complexity of this time-lag effect is, in fact, one of the major problems involved in attempting to relate distress to performance.

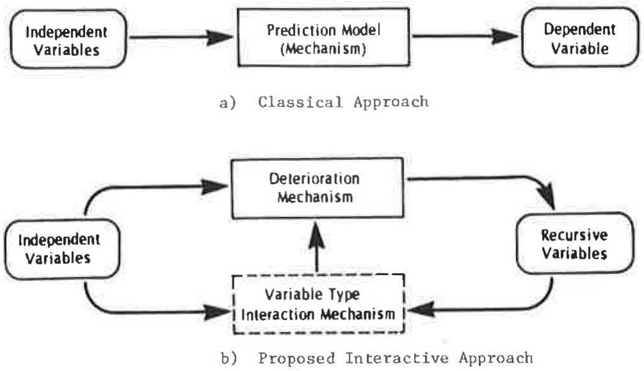
The use of roughness alone to estimate serviceability seems more logical since the user's perception of pavement serviceability is dominated by riding comfort (5). Therefore, by attempting to relate roughness to different combinations of the various amounts of distress measured during various periods of a pavement's life, the serviceability-age profile or performance of a pavement can be more realistically related to distress.

Interactive Modeling Approach

The design-check or classical approach to modeling pavement behavior has resulted in a segmentation of the pavement deterioration process into a number of independent, distress-mode-specific submechanisms. In general, most of the present concepts of pavement behavior have evolved from two basic assumptions: (a) each form of distress pertains to a specific distress mechanism, and each is independent of other mechanisms at work and (b) the amount of each type of distress is dependent only on the levels of certain nonbehavioral variables or stimuli and is therefore unrelated to the behavioral history of the pavement.

The assumption of distress-mode independence is an oversimplification of the deterioration process. Losses in both load transfer and resistance to water infiltration, for example, result from the formation of cracks in the surface layer of a pavement. It is more likely that other distress mechanisms would be quite sensitive to such an occurrence. Indeed, the rates of both crack production and crack propagation

Figure 3. Conceptual framework of pavement behavior models.



are, most likely, closely related to the cracking history of a pavement.

In order to develop quantitative relationships between distress and performance, a more comprehensive theory of pavement behavior has been developed. The proposed modeling approach is based on the premise that one overall deterioration mechanism controls the appearance of all types of pavement distress. The theory advanced in this paper, in contrast to the assumptions underlying the classical approach, is based on the following hypotheses:

1. Different distress submechanisms are interrelated through time. The amount of a specific type of distress observed at one point in time may affect the occurrence or level of another type of distress at some future time.
2. Pavement behavior elements are recursive in that the characteristics that are likely to appear in the future are dependent on the history of those same characteristics.

The variables used in modeling pavement behavior tend to fall into two categories, according to the mathematical structure of the relationships. Under the classical approach, variables can be grouped as either independent or dependent variables. This dichotomy has been modified in the proposed modeling approach into independent and recursive variables. The following are common examples of variables used to model pavement behavior:

1. Traffic loads,
2. Environment (drainage, frost, temperature, etc.),
3. Pavement structure (layer thicknesses and orientation),
4. Pavement type (layer and material properties),
5. Structural capacity,
6. Pavement condition (levels of distress), and
7. Pavement roughness.

In the classical approach, variables 1-5 would be classified as independent and variables 6 and 7 as dependent. In the proposed approach, however, variables 1-3 would be classified as independent and variables 4-7 as recursive. Of the three variables sharing the common independent classification, traffic loads and environment fall beyond the designer's control, and pavement structure is generally stable with time.

Under the classical approach, structural capacity is usually classified as an independent variable because of its general relationship to materials and layer properties and construction practices and because it has been used to predict performance. However, some pavement management systems include structural capacity as a condition parameter in priority programming (6).

Under the proposed approach, many behavior variables have been reclassified as recursive. A recursive variable is one whose present behavior is directly related to its historical behavior, thereby implying a time-dependent nature. Although the pavement type would generally be designer controlled, there is no reason to suspect that this should in general be stable with time, since layer properties can change with time and environment. The remaining recursive variables are uncontrollable and usually unstable with time.

Figure 3 compares the conceptual frameworks for the classical and interactive approaches to modeling pavement behavior. By the classical approach illustrated in Figure 3a, estimates of a dependent variable (behavior) are derived from specific values of the relevant independent variables through a simple set of relationships contained in the prediction model. The two assumptions underlying the classical approach are evident in this formulation in that only one form of behavior is considered per model and the prediction process is independent of time.

Figure 3b presents the hypothesized approach to modeling pavement behavior. The values of the recursive variables that include the various types of distress and roughness are predicted iteratively and simultaneously. All predictions are recycled back into the deterioration mechanism, by which they are used along with all past known and predicted values to make predictions further into the future. In this manner, predicted time profiles are built up for each behavior variable.

Part of the mechanism indicated by Figure 3b is a variable type of interaction mechanism that should be incorporated into the overall mechanism, instead of being shown as a separate process. However, it is identified in this manner to emphasize the potential importance of variable interaction. The predicted behavior interacts with the independent variables to modify their effect in the deterioration process.

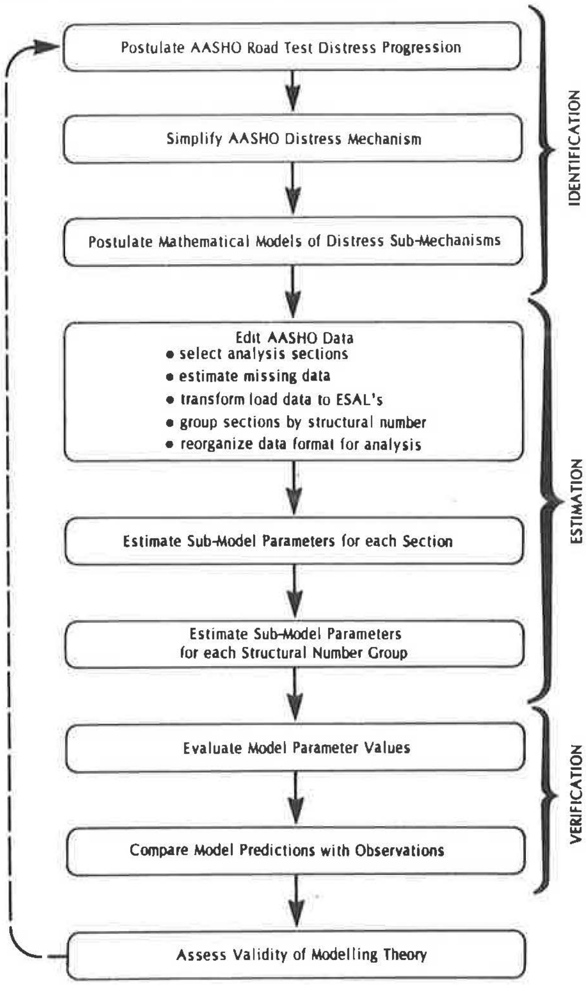
For example, once a certain level of cracking has been predicted, the effect of the same loads will increase pavement deterioration as the load-bearing capacity of the pavement structure is decreased. In another example, when roughness increases, the effect of the same traffic loads on the pavement structure may be magnified depending on the vehicles' shock-absorbing properties and operating speeds.

This modeling concept has great potential for the development of distress-performance relationships since the roughness-age profile (the most significant measure of performance from the user's perspective) can be predicted concurrently with distress. This concept also has the capability of incorporating a Bayesian approach to update predictions and to modify initial model parameters or even model structure as new information (distress and roughness measurements) becomes available.

PRELIMINARY INVESTIGATION

A preliminary investigation (4) was undertaken to test the workability of the proposed modeling approach for developing pavement distress-performance relationships. The analysis, conducted by using actual test-road data, has produced a rational (although not statistically strong) relationship between pavement distress and performance.

Figure 4. Overview of preliminary investigation.



Overview of Investigation

In order to test the effectiveness of the proposed modeling approach, a model-building procedure similar to the one recommended by Box and Jenkins (7) for modeling stochastic processes was adopted. The three main phases of this approach are illustrated in Figure 4.

In the identification stage, relationships were postulated and a general structure given to the model. This involved postulating a distress mechanism for the data used (from the AASHO Road Test), organizing and simplifying this mechanism into components, and postulating mathematical models for each component.

The estimation stage involved estimating values for the parameters of the models postulated. Prior to this, the data used for estimation had been reorganized for compatibility with both the estimation technique and the model formulations.

The verification stage involved an assessment of the estimated parameters to test the postulated relationships for reasonableness. In the normal course of this process, the pavement behavior would be predicted by using the estimated model and compared with the actual behavior measured by a statistical analysis. If the model were found to be statistically unacceptable, the relationships could be repostulated, new mathematical models formulated, and the process repeated. This iterative approach would continue until the models estimated resulted in acceptable predictions.

Ideally, verification would not only involve testing for goodness of fit for the data used in estimation but also testing the general applicability by applying the estimated model to an independent set of data. Such verification was not possible in this investigation because the data used were obtained from the only known source compatible with the modeling approach.

Data Requirements

Time-delay effects and distress-mode interaction have been incorporated into the proposed modeling approach. This implies the use of stochastic techniques to model the deterioration process. The stochastic modeling approach, however, requires a comprehensive data base in which observations are in the form of time series. The only existing data base that meets these requirements is the AASHO Road Test data.

A partial factorial experiment of pavement structures, materials, and load configuration was designed, constructed, and tested near Ottawa, Illinois, in the 1950s. Loads were applied to the test sections for two years, and several types of measurements were taken at two-week intervals throughout the loading period. The pavement sections in the test's six traffic loops were all constructed on the same, uniform, preplaced embankment material in an attempt to eliminate subgrade and environmental variations.

For this investigation, only flexible pavements were considered, although the main factorial design included both flexible and rigid structures. In the main factorial design, six levels of surface thickness, four levels of granular base thickness, five levels of subbase thickness, and 11 load configurations (including no load) were used.

Of the data available from the AASHO Road Test, only certain variables were considered in the model formulation. Pavement structure (surface, base, and subbase thicknesses) and loading (cumulative repetitions by configuration) were the only independent variables used. The recursive variables included cracking (classes 1, and 2, and 3), rut depth, and roughness in terms of slope variance (8). Measurements for the three recursive variables were taken every two weeks up to a maximum of 55 measurements.

Three important requirements of time series must be fulfilled before stochastic modeling is possible. First, the time intervals between measurements must be the same for the entire series. The AASHO Road Test data are apparently unique in the satisfaction of this requirement. Second, with three recursive variables in the analysis, each of which may be defined at several different levels, it is statistically desirable to have at least 30 observations in each time series. Again, the AASHO Road Test data are unique in the comprehensive nature of the time series. The third requirement of time series is completeness, which relates to the first requirement in that, if an observation is missing, the time intervals are not consistent. This is the most common and serious problem with time-series data in every discipline. The AASHO Road Test data are quite deficient in this respect. Because analytical techniques to overcome this problem are apparently not available, the only recourse was to estimate values for missing observations whenever this situation arose.

The environmental and structural capacity variables, although available, were excluded from the analysis for the following reason. The number of

Figure 5. Distress progression postulated for AASHO road test data.

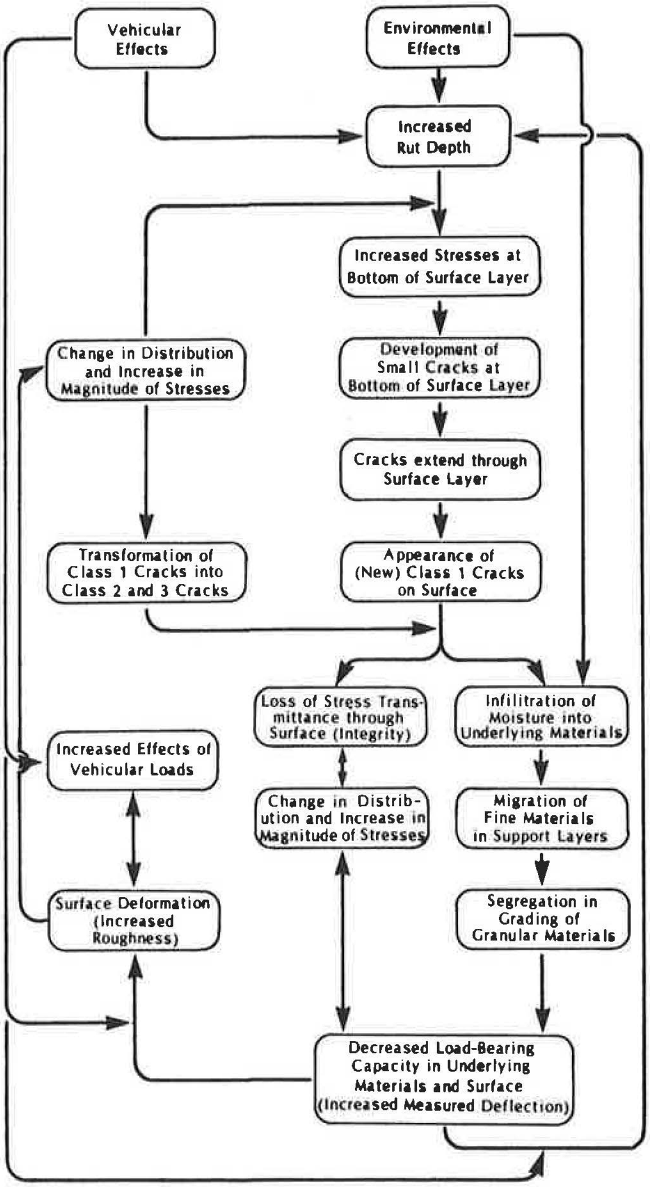
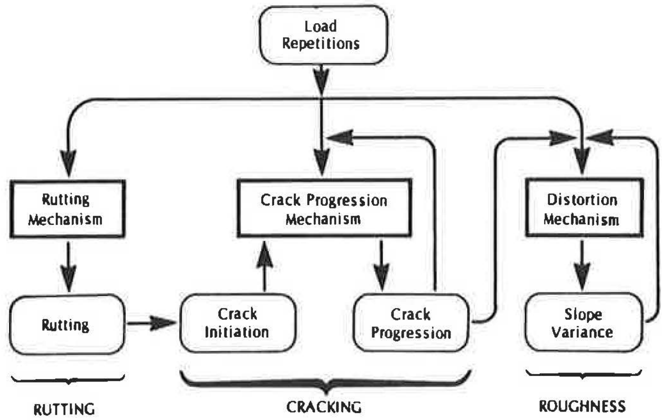


Figure 6. Simplified AASHO distress mechanism.



observations available in the individual time series restricts the number of variables that can be modeled and still maintain statistical significance. Since one type of subgrade and set of environmental conditions prevailed for all sections, the exclusion of environmental and structural capacity variables would add less unexplained variation to the models estimated than the exclusion of one or more of the variables chosen.

Model Formulation

The first step in the investigation was to postulate a behavioral model of the AASHO Road Test flexible pavements. Figure 5 presents the complex deterioration model postulated. It has been suggested that the principal mechanism attributable to the failure of AASHO Road Test test sections is fatigue (9). It has also been perceived that crack initiation and progression could generally be associated with a certain degree of rutting. These ideas have been incorporated into Figure 5 as the main cause of crack initiation.

It has also been postulated that the progression of cracking results from changes in stress distribution and a loss of the load-bearing capacity of underlying materials through moisture infiltration. The development of surface roughness is attributed to this structural weakening. The crack-progression cycle relates to the further increase in the severity of existing cracks. However, the change in stresses in the pavement structure is also identified as a factor affecting the further initiation of cracking. This formulation combines the modeling approach presented earlier with the existing knowledge of flexible-pavement deterioration mechanisms.

The general distress progression presented in Figure 5 contains many elements that cannot be substantiated by the field measurements available. Stresses were only measured in rigid pavements, and the migration of fine material and segregated grading are phenomena that could not be monitored nondestructively. For these reasons, this deterioration mechanism was simplified into three major submechanisms, as shown in Figure 6.

Under this formulation, the rutting submechanism uses only load repetitions to produce ruts. Cracks are, in turn, initiated according to the level of rutting reached. The presence of cracks then affects the pavement structure so that continued loading propagates these cracks, according to the cracking submechanism. The progression of these cracks also continues to magnify the effect of loading on crack progression. Progressive cracking also interacts with loading to produce roughness (in terms of slope variance) according to the distortion submechanism. Increasing roughness also tends to interact with the load repetitions within the distortion submechanism to further increase roughness.

Mathematical models were formulated for each submechanism. These formulations were applied to pavements in five different structural number (SN) groups. Hence, the 20 parameters to be estimated actually numbered 100, although this number was, in reality, less because of a lack of observations in certain variable-level categories.

Since load is known to be the major cause or stimulant of rutting, the rutting submodel used cumulative equivalent single-axle loads (ESALs) to predict the rut-depth time series for discrete time intervals as shown below:

$$R_t = a + b L_t^2 \quad (1)$$

where

R_t = rut depth at time t,
L_t = log of equivalent 80-kN single-axle loads accumulated by time t, and
a,b = model parameters (to be estimated).

Several transformations of the loading variables were examined; however, the second-degree polynomial expression of Equation 1 represented the best statistical fit.

The cracking submodel had three components. The first component involved a postulation of the cumulative loads (LI) required to initiate cracking for each pavement structure group. Cracking was defined as the total of class 1, 2, and 3 cracking (8). Generally, it was found that class 1 cracks in the AASHO Road Test data corresponded to total cracking between 0 and 5 percent and, hence, classes 2 and 3 corresponded to total cracking values greater than 5 percent. Equations 2, 3, and 4 below are the discrete mathematical formulations corresponding to the three levels of total cracking:

$$C_t = 0 \quad \forall\{L_t < LI\} \tag{2}$$

$$\nabla C_t = c_1 + d\,R_t + e\;\nabla L_t \quad \forall\{0 < C_t < 5\%\} \tag{3}$$

$$\nabla C_t = c_2 + e_2\,\nabla L_t \quad \forall\{5\% \leqslant C_t\} \tag{4}$$

where

C_t = cracking at time t,
∇ = first-order differencing operator ($\nabla C_t = C_{t+1} - C_t$),
c,d,e = model parameters (to be estimated), and
LI = the cumulative loads required to initiate cracking.

Rutting was excluded from Equation 4 because it was felt that the presence of class 2 and class 3 cracking has more influence on the rate of increase in cracking than does rut depth. In both Equations 3 and 4 the rate of crack progression is assumed to depend on the rate of load accumulation. The faster the rate of loading, the less likely is the pavement to recover from the accumulated damage and, therefore, the more likely cracking is to occur.

Equations 3 and 4 are difference equations. These are simply discrete forms of differential equations in which the differencing operator (∇) replaces the differential operator (D).

The roughness submodel has six components of the same formulation. Different parameter estimates were required for each combination i of cracking interval and load interval. Equation 5 below represents the difference-equation form of the roughness submodel:

$$\nabla SV_t = f_i + g_i\,SV_t \tag{5}$$

where

SV_t = slope variance (roughness) at time t,
f,g = model parameters (to be estimated), and
i = combination of load level (L_t < LI; LI $\leqslant L_t$) and cracking level (C_t = 0; 0 < C_t < 5%; 5% $\leqslant C_t$).

This formulation highlights the interactive nature of the distress mechanism and, since roughness is the most important indicator of riding comfort and hence serviceability, the interactive nature of the distress-performance relationship is illustrated. Although not theoretically meaningful according to the formulation, parameters for Equation 5 must also be estimated for cracking between 0 and 5 percent and cracking greater than 5 percent when load L_t is less than LI and also for the case where cracking is 0 but load L_t is greater than or equal to LI. Since the LI for each SN group is simply the average of the cumulative ESALs at crack initiation for pavement sections in each group, the value of LI computed in this manner represents the theoretical value only about 50 percent of the time.

Like differential equations, difference equations can be integrated or solved to remove all rate variables. When Equations 3, 4, and 5 are integrated (4), the recursive nature of the variables is illustrated.

In addition to the difference-equation formulation of the cracking submodel, a crack-state transition model was also developed and tested as a second iteration of the model-building process described in Figure 4. Three states of cracking (0 = no cracking, 1 = class 1 cracking, and 2 = class 2 and class 3 cracking) and five state transitions (0 to 1, 0 to 2, 1 to 1, 1 to 2, and 2 to 2) were used in this model. The model is based on initiation and transition parameters. The initiation parameters include the load required for initial transition to states 1 or 2 from state 0 and the associated initial area change. The transition parameters describe the extra load and area changes for transitions from states 0 to 1, 1 to 2, and 0 to 2 after crack initiation. The model operates by adding subareas of the pavement surface to cumulative sums of the areas in cracking states 1 or 2 as determined by the accumulated loads. A more detailed account of this model is given by Smeaton (4).

Parameter Estimation

Figure 4 shows three major activities in the estimation of the model parameters. Before these could actually be estimated, however, it was necessary to edit and reorganize the AASHO Road Test data available into a format compatible with the form of the equations being considered and the estimation technique used. The first task was to select suitable pavement sections for analysis. Only main-factorial, flexible-pavement sections that had been subjected to traffic were considered. At least 30 observations were required for each time series. Many sections were screened out and excluded from the analysis because of inconsistencies, the performance of maintenance, the omission of cracking data in the AASHO Road Test data system 4100-F, or the occurrence of gaps of more than three consecutive observations prior to the 30th measurement interval. Of the 284 main-factorial, trafficked, flexible-pavement sections available, 211 were excluded, leaving 73 sections for the analysis. These are listed with each section's last observation number in Table 1.

The second task in the editing operation was to estimate the missing observations. The slope variance and rut-depth time series were taken from AASHO Road Test data system 7322-D. These time series are riddled with missing observations in every section. Approximately 2 percent of the rut-depth time series was missing, whereas roughly 11 percent of the slope variance time series was missing. In an attempt to maintain consistency in the analysis, it was necessary to estimate the missing observations as objectively as possible. In the apparent absence of such mathematical or statistical estimation techniques, a least-squares method (4) was developed and applied to each time series.

Table 1. AASHO test sections selected for analysis.

Section	Last Observation	Section	Last Observation	Section	Last Observation	Section	Last Observation	Section	Last Observation
Loop 2		Loop 4		629	39	445	38	316	39
				630	41	446	55	308	38
742	38	617	32	616	35	475	39	327	52
758	55	618	38	591	55	483	37	328	45
738	45	624	34	592	55	484	37	313	35
746	55	601	55			447	55	314	39
750	55	602	40	Loop 5		448	42	331	45
		604	37					265	55
Loop 3		575	37	441	55	Loop 6		298	35
		576	40	442	41			255	31
131	37	577	55	454	37	300	35	256	55
155	49	578	55	426	31	318	38	257	55
156	38	625	55	477	55	254	40	301	55
122	36	626	55	478	42	268	45	271	55
124	39	622	40	479	55	309	45	272	55
140	54	632	45	480	42	310	45	311	38
		593	37	469	41	262	37	312	55
		594	43					333	38

Table 2. Parameter estimates.

Load Interval		Estimated Submodel Parameters									
		Rutting		Cracking				Roughness			
								$L < LI_i$		$LI_i \leq L$	
SN Group	Cracking Interval	a	b	LI_i	c	d	e	f	g	f	g
	C = 0							25.869	−0.609	53.744	−0.835
1	0 < C < 5%	−0.2516	0.005 00	11.3315	0.062	3.585	−13.944	53.343	−0.719	64.031	0.115
	5% ≤ C				−0.914		33.109	167.098	−0.918	298.571	−1.192
	C = 0							23.388	−0.558	40.396	−0.818
2	0 < C < 5%	−0.5047	0.005 95	13.2547	−0.025	2.155	−8.617	67.397	−0.727	94.569	−0.426
	5% ≤ C				0.417		17.453	–	–	179.698	−0.756
	C = 0							16.010	−0.371	70.130	−0.997
3	0 < C < 5%	−0.5150	0.005 72	13.5521	0.534	1.693	−21.284	127.099	−1.041	149.881	−1.059
	5% ≤ C				−0.762		31.354	128.638	−0.590	160.658	−0.512
	C = 0							16.814	−0.421	47.892	−0.411
4	0 < C < 5%	−0.5151	0.005 44	14.0156	−0.916	1.078	9.259	70.452	−1.187	30.083	−0.024
	5% ≤ C				–		–	–	–	346.153	−1.331
	C = 0							13.582	−0.417	24.719	−0.536
5	0 < C < 5%	−0.5214	0.004 67	14.8759	1.994	−1.670	−11.846	37.482	−0.606	51.848	−0.534
	5% ≤ C				−1.133		41.896	–	–	135.886	−0.722

The third task involved transforming portions of the data in order to reduce the number of variables in the analysis. The pavement layer thicknesses were converted to SNs and the sections grouped into five categories according to the SN intervals presented below:

Interval	SN Range
1	$SN_1 < 3.25$
2	$3.25 \leq SN_2 < 3.75$
3	$3.75 \leq SN_3 < 4.00$
4	$4.00 \leq SN_4 < 4.50$
5	$4.50 \leq SN_5$

SNs were calculated by applying layer coefficients to the surface (D_1), base (D_2), and subbase (D_3) thicknesses, using the following equation (8):

$$SN = 0.44D_1 + 0.14D_2 + 0.11D_3 \quad (6)$$

In order to eliminate axle load as a variable, all load accumulations for each axle load (L, measured in kips) were converted to equivalent 80-kN (18-kip) single-axle loads by using the following formulas for equivalency factors k (10):

$$\text{Single axles: } k = (L/18)^4 \quad (7)$$

$$\text{Tandem axles: } k = (L/33)^4 \quad (8)$$

A logarithmic transformation was also applied to every ESAL time series in the analysis.

The final task in editing the data was to calculate the differenced (Equations 3, 4, and 5) variables (cracking, slope variance, and load) and to group the data according to SN and observed levels of cracking and loads to initial cracking (LI).

The parameters in Equations 1 to 5 were estimated for each AASHO Road Test section considered by using multiple linear regression. A weighted average was obtained for each parameter of each SN group by multiplying the parameter obtained for each section by the number of observations used to estimate it, summing these products by SN group, and then dividing by the total number of observations in each group. The parameters estimated for each equation and SN group are presented in Table 2.

A fairly good relationship (R^2 = 0.66) was developed between LI and SN. When the averaged values for LI (see Table 2) and SN for each SN group i were regressed, the equation presented below was obtained:

$$LI = 7.7423 + 1.4620\ SN,\ n = 5,\ R^2 = 0.96 \quad (9)$$

This expression verifies the validity of the crack-initiation portion of the model.

Model Evaluation

The final stage of the investigation was to assess the reasonableness of the relationships postulated by considering the parameters estimated in the analysis.

The model developed in this preliminary investigation is not statistically strong; however, several patterns appear in the parameter estimates presented in Table 2 that support the validity of the proposed relationships and modeling approach. A detailed presentation of the model statistics may be found in Smeaton (4).

The rutting submodel was found to have a very strong statistical significance, as could be expected. Both the individual section submodels and the averaged SN group submodels had relatively high multiple correlation coefficients. The slopes of the submodels (b) were quite uniform for all SN groups. However, the negative intercepts indicate that a discontinuity exists in all rutting time series. This was found to correspond to the first spring thaw of the test, when the temperature jumped from roughly 14°C (57°F) to 26°C (79°F) in a short period of time.

The cracking submodel appears to be the weakest statistical link in the interactive model. A great deal of variation can be attributed to inconsistencies in the AASHO Road Test cracking data. Many cracking time series showed negative cracking rates (decreasing cracking) with no reports of maintenance. The crack-state transition formulation detailed in Smeaton (4) did not improve on the statistical performance of the difference-equation formulation (Equations 3 to 5), but the parameter estimates showed without exception that as the severity of cracking increases the rate of cracking also increases and, hence, cracking is a recursive variable.

A comparison of the load-rate coefficients e (of the difference-equation formulation presented in Table 2) between the two cracking levels indicates that as the amount of cracking increases the load rate has more effect on the rate at which cracking increases. This supports the hypothesis that cracking is recursive and interacts with other variables. The statistical analysis (4) also showed that the variation not explained by the model was significantly less for cracking in excess of 5 percent than for cracking between 0 and 5 percent. This suggests that deterioration accelerates in a fairly stable fashion once the cracking level has reached 5 percent.

The roughness submodel parameters f and g presented in Table 2 tend to confirm the postulated relationship quite dramatically. A very consistent relationship is evident between the level of cracking and the value of the constant term in each equation. As the amount of cracking increases, the level of roughness also increases. The same trend is evident in every model for increasing axle loads. As the cumulative ESALs increase, the level of roughness also increases. No such relationship appears to exist for SN.

The slopes g of these models are generally quite stable, indicating a relatively constant rate of increase in roughness. This is probably a result of the short-term variation present in the slope variance data that can at least partly be attributed to uncontrollable noise in the measurement techniques.

In general, the roughness submodel parameters appear to be quite rational and fairly reliable. The results of this investigation are quite encouraging, although by no means conclusive. A great deal of research effort is still required before useful relationships between pavement distress and performance are operational.

SUMMARY AND AREAS FOR FURTHER INVESTIGATION

Although this investigation did not produce prediction models that could be immediately applied in a working sense, the usefulness of applying the proposed modeling theory to the development of objective relationships between pavement distress and performance has been illustrated. Specifically, the results of this study support the hypothesis that different forms of pavement distress are interdependent through time and depend not only on independent variables or stimuli but also on their historical behavior.

The following is a list of areas of refinement and further investigation needs arising from this study.

1. It would be desirable to reanalyze the AASHO Road Test cracking data to determine the model's sensitivity to the definitions of cracking levels. Much of the variation in these data was in the range between 0 and 5 percent. Also, a more comprehensive investigation of the loads to crack initiation that uses a larger portion of the available data could provide better insight into the crack-initiation phenomenon. In close relation to this, the AASHO Road Test sections that did not develop cracking should also be examined to perhaps provide more insight into the deterioration process.
2. The distress mechanism presented in Figure 6 should be modified to incorporate structural capacity. Time series of Benkelman beam deflections were measured at the AASHO Road Test by using the same measurement intervals. This modification could provide further insight into the mechanics of the deterioration process.
3. A second level of verification should be conducted in future analyses by using the sections excluded from analysis. This would provide a better indication of model reliability.
4. Time series analysis could provide a more reliable and informative approach to grouping sections, in that parameters describing specific behavior characteristics could be used to derive weighting functions for averaging estimated parameters within SN groups. The weighting procedure used in this study may have been too insensitive to individual patterns of behavior, since these patterns were often quite different within SN groups.
5. The least-squares estimation (regression) technique used was applied to each submodel independently. However, a simultaneous estimation of these parameters, as is possible with a two-stage least-squares approach, would maintain greater statistical consistency between submodel parameters. Stochastic modeling techniques also have considerable scope in the proposed behavior modeling approach because of the apparent recursive nature of the distress variables.
6. The main criticism of the AASHO Road Test has been the accelerated nature of its load applications. Another is that the short section lengths (100-200 ft) do not simulate representative behavior patterns that occur in practice. These, along with other considerations, limit the

applicability of the AASHO Road Test data. One of the findings of this preliminary study is the need for a more comprehensive, realistic, and consistent data base before useful relationships between distress and performance are analytically possible. This can be accomplished with the cooperation of state, provincial, and regional highway agencies through improved behavior monitoring and documentation practices. Specifically, highway engineers should attempt to schedule periodic evaluation on homogeneous pavement sections at regular intervals of time. The technology now exists to measure and make computerized, "manageable" records of a number of pavement behavior parameters continuously and simultaneously at great savings in both time and money. Seasonal effects play an important role in characterizing pavement behavior patterns and, therefore, it is desirable to monitor behavior more than once a year on each pavement section.

7. The measurement of cracking is still a highly subjective operation, as evidenced by the numerous inconsistencies and high variability of the AASHO Road Test cracking data. Not only is there a need for more objective crack-measurement techniques, but in addition the definition of cracking should reflect the physical behavior, not the hypothesized cause or mechanism. There is also a need for uniformity among different agencies in the definitions of cracking and other forms of pavement behavior. Such consistency and cooperation would aid in the development of a more comprehensive data base from which pavement distress-performance relationships could ultimately be developed.

8. The modeling approach presented in this paper is equally applicable to pavement types other than flexible pavements. Rigid pavements and also new types such as sulfur-extended asphalt pavements should be included in future investigations.

9. In considering pavement deterioration as a stochastic process, the performance of maintenance is an intervention in that process. Intervention analysis is a statistical technique for determining whether a known intervention significantly alters the behavior of a stochastic process. This is a potential means of evaluating the effectiveness of various types of maintenance or rehabilitation strategies.

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Requirements for Reliable Predictive Pavement Models

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The general requirements for reliable pavement prediction models are presented. Performance models are essential for efficient management of pavements. Experience has shown that they can best be derived from a data base of in-service pavements. The major requirements of a reliable model for predicting performance, herein defined as serviceability index and distress occurrence over time, are (a) an adequate data base built from in-service pavements, (b) the inclusion of all variables (including mechanistic variables) that significantly affect performance, and (c) an adequate functional form of the model that considers shape, nonlinearities, and interactions; meets boundary conditions; and provides reasonable sensitivity of variables. The model must also meet statistical criteria for precision (e.g., error of prediction, R^2, and regression coefficients).

This paper describes the requirements and general development of reliable pavement performance models derived from a data base of in-service pavement information. Pavement management requires the use of performance models for design of new pavements as well as the maintenance and rehabilitation of older pavements. Data from in-service pavements are also needed for use in establishing the validity or in calibrating predictive design models derived from mechanistic concepts. The resulting designs will only be as reliable as the models used in their development; thus, their accuracy and capability are very important.

Predictive performance models can conceivably be mechanistic in nature when the relationship between the dependent and independent variables is exactly known (e.g., F = ma). However, the prediction of the present serviceability index (PSI), pavement condition index (PCI) (1), and distress history depend on many variables in extremely complex ways. Thus, the only practical predictive model that can be developed is an empirical model (or semiempirical model with some mechanistic input) based on measured data. Multiple regression techniques are commonly used to develop empirical predictive models. This paper is limited to the development of linear regression models and to the requirements of

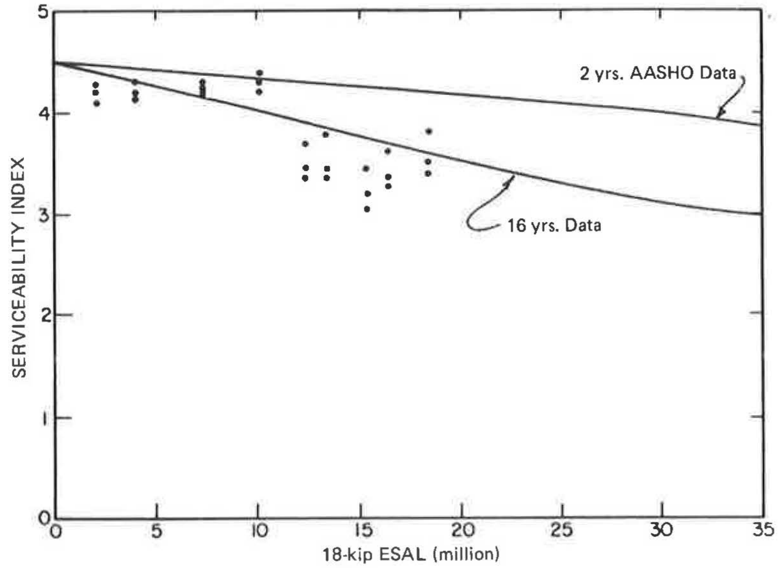


Figure 1. Illustration of original prediction model based on 2 years' data and new prediction model based on 16 years' data.

reliable models derived from a data base of information from in-service pavements.

Typical linear regression models take the following general form:

$$\hat{Y} = a_0 + a_1 X_1 + a_2 X_2 + \dots a_n X_n \quad (1)$$

where

$\hat{Y}$ = estimated value of the dependent variable Y (performance indicator such as PSI, PCI, or distress),

$X_1, X_2, \dots, X_n$ = value of the independent variables (such as layer thickness, material properties, climatic parameters, and traffic factors), and

$a_0, a_1, a_2, \dots, a_n$ = parameters of the model estimated by regression.

The difference between the measured value Y and the estimated value $\hat{Y}$ from Equation 1 for each data case is called a residual or the error in prediction: Residual = $Y - \hat{Y}$. The regression procedure involves the selection of a_0, a_1, a_2, ... so that the sum of the squared residuals over all the data is minimized. Since the sum of squared residuals is minimized, the process is called least squares. Thus, no other line is closer to all the data points. This paper is not intended to provide a detailed statistical description of regression analysis but to discuss key practical considerations in developing predictive models.

The major requirements of a reliable model for predicting performance include an adequate data base, the inclusion of all significant variables that affect performance, an adequate functional form of the model, and the satisfaction of statistical criteria concerning the precision of the model.

ADEQUATE DATA BASE

The most important consideration in developing a reliable performance model is the building of an adequate data base. First and foremost, the data base must be a representation of the overall pavement network that the model is being developed to represent. Next, the data collected must be measured accurately and without bias. Finally, there must be a sufficient number of data cases so that practical and statistical requirements can be satisfied.

Representative Sample

A predictive model may be needed for use in pavement design or rehabilitation over a given geographic area (e.g., a state or nation). Since it is generally impossible to physically measure data from every pavement located in the geographical area, a sample of the data must be selected. Regression analysis is commonly performed on a data sample from which the overall population (or pavement network) parameters can be estimated. The data sample collected, therefore, must be representative of the general geographic region as far as materials, designs, traffic, soils, climate, and age (varying up to the desired design life) go. This fundamental concept of an adequate data base shows that the results from a single road test provide only limited information on which to base a performance model. For example, many persons have criticized the nationwide use of the models derived at the AASHO Road Test under very limited conditions. In fact, analyses conducted on 16 years of data from 25 sections of the Road Test on I-80 showed that the original equations overpredicted performance, as illustrated in Figure 1 (2).

The data base should ideally include data cases (e.g., projects) that contain a sufficient range of each of the variables. For example, if a range of surface thickness is not included, it is impossible to include that variable in the model and to determine its relative influence on performance. A data base constructed as a data factorial with three or more levels of each variable provides a balanced set of data from which to develop a broad-based model, as illustrated in Figure 2. However, because of the nature of in-service pavement data (e.g., messy data), this is usually impossible to completely accomplish. It is helpful, however, to at least use a factorial design in the initial planning of the data collection to provide a data bank as broadly based as possible.

Reliable Data

The overall data to be collected can be divided into

Figure 2. Illustration of factorial data sample to provide balanced information to develop predictive models.

CLIMATE	DESIGN (SLAB THICK-CM)	SUBGRADE / AGE, YEARS: COARSE GRAINED SOIL < 10	COARSE GRAINED SOIL 10-19	COARSE GRAINED SOIL ≥ 20	FINE GRAINED SOIL < 10	FINE GRAINED SOIL 10-19	FINE GRAINED SOIL ≥ 20
WET-NONFREEZE	15						
	20						
	25						
WET-FREEZE	15			1 or more pvt sect.			
	20						
	25						
DRY-NONFREEZE	15						
	20						
	25						
DRY-FREEZE	15						
	20						
	25						

field information and historical information. The field data are obtained from surveys and measurements on each project. The historical data are obtained from agency records (e.g., traffic, materials, climate, and construction). Sometimes portions of the historical data were never collected on a given project or, if collected, were thrown away or lost. Care must be taken to assure the accuracy of the data obtained from historical records. Field data must be obtained by using carefully developed procedures. In the past, a few agencies have developed field data collection procedures (3-6). Comprehensive procedures have recently been developed and verified for both historical and field data for airfield pavements (1), highway pavements (7-9), and road and street pavements (10). Survey crews can be sufficiently trained to be consistent in data gathering. Equipment must be maintained and kept calibrated so that the measurements do not change over time.

Sufficient Amount of Data

The development of a reliable model requires the collection of a sufficient number of data cases. A case is the basic unit of analysis for which data have been obtained. In terms of pavement engineering, a case could be a single test section or an entire construction project. Each case is composed of one data value for each of the several variables under consideration. For example, a data case could consist of the following data values from a single section of pavement: PSI, alligator cracking, rutting, surface thickness, base thickness, California bearing ratio (CBR) of base, CBR of subgrade, average number of freeze-thaw cycles, deflection, and total accumulated 80-kN (18-kip) equivalent single-axle loads. This set of information would define a single data case.

The actual length of pavement that makes up the case must be carefully defined. For example, a case has been defined in NCHRP Project 1-19 as a uniform section of pavement that has the following uniform characteristics along its length (7): structural design, joint and reinforcement design, proportion of truck traffic, number of lanes, subgrade conditions, construction by same contractor, open to traffic same year, pavement materials, and maintenance applied. In most instances, the uniform section will correspond to a regular construction project. This procedure avoids the problem of having widely varying results within a given data case, which ultimately leads to masking the true effect of the variables involved.

Analysis of field data for a given uniform section in NCHRP Project 1-19 showed that, for individual distress types, a minimum of approximately 10 percent of the uniform section length should be measured. A length of 0.16 km/1.6 km (0.1 mile/mile) is the recommended stratified sampling plan.

It is also important to obtain some replicate data cases. A replicate data case is two or more pavements identically constructed (as far as is known), placed on the same subgrade, and subject to the same climate and traffic. Any difference in performance between the two gives an indication of pure error. The value of this estimate is discussed under statistical criteria.

INCLUSION OF VARIABLES

Identification of Variables

Every possible variable that may affect pavement performance should be considered initially. This list will typically be very large. These variables are then divided into groups such as the following:

1. Data that can be directly measured within acceptable cost and time constraints;
2. Data that are measurable but too expensive or time consuming for regular collection;
3. Data that are not available from records or that cannot currently be measured;
4. Data that are available from historical records on design, construction, performance, traffic, maintenance, and climate; and
5. Data that can be computed or estimated based on the above types of data.

Available time and resources will not generally be sufficient to collect all desired data. However, this process will reveal the major deficiencies and limitations in the models by pointing out the variables that are not included in the model. A practical assessment of which variables can be collected must be made. The question that should be continually asked is, How will the deletion of this variable limit the usefulness of the model? Sometimes, when the direct measurement of a variable is not feasible, another variable that correlates highly with the other variable can be included.

One way to assess the scope of the model is to categorize the variables included under major topics that are known to affect performance, such as layer material properties, subgrade characterization, layer geometry, climate, traffic, jointing, construction (e.g., quality control), maintenance, and drainage. If each of these general topics is adequately represented by one or more variables, then the model should contain most of the important variables known to affect performance.

Mechanistic variables that can be computed by using various pavement structural programs (e.g., elastic layer, finite element) can add significantly to the reliability and utility of the prediction model. For example, in the development of prediction models for asphalt pavements, the strain and stress in various layers could be computed for a given load and temperature and used as a variable in regression. The ratio of tensile stress to strength of the concrete slab can lead to a much improved model.

Variable Selection

The selection of the best regression model to fit a given sample of data requires extensive knowledge about (a) the problem at hand and the measured data and (b) a regression analysis program. Several statistical procedures exist for selecting variables in regression. Several methods are discussed and compared by Draper and Smith (11). The stepwise regression procedure that employs the least-squares method is believed to give the best selection of independent variables. As explained in Draper and Smith (11), the stepwise regression procedure starts with the simple correlation matrix and enters into regression the independent variable (X) most highly correlated with the dependent variable (Y) (e.g., PSI, distress). By using partial correlation coefficients, it then selects (as the next variable to enter regression) that X variable whose partial correlation with the response Y is highest and so on. The procedure reexamines "at every stage of the regression the variables incorporated into the model in previous stages" (11). The procedure does this by testing every variable at each stage as if it entered last and by checking its contribution by means of the partial F test. The process is stopped when essentially no additional variable significantly improves the precision of the model.

A few excellent computerized statistical-analysis packages are available. One of the most well-documented packages and easiest to use for regression and many other analyses is the Statistical Package for the Social Sciences (SPSS) (12).

FUNCTIONAL FORM OF MODEL

The functional form of the model, or the way in which the variables are arranged, has a great effect on its reliability. The functional form must be established through careful thought about the actual relationships between the variables and the plotting of available data (Y versus all X's). The functional form cannot be left to the computer to establish; the researcher must be thoroughly familiar with the data. The reliability of the functional form of the predictive model can be assessed through considering the linearity and additivity of the variables, the boundary conditions, and a sensitivity analysis.

Linearity and Additivity of Variables

Basic linear regression requires that the relationships among the variables are linear and additive as shown by Equation 1. Thus, the relationship between Y and X_i is assumed to be linear and the combined effects of the X's are additive. These assumptions are not generally true for pavement variables. There are, however, methods to handle nonlinear and nonadditive relationships. The three most-often-used methods are briefly described:

1. Transform the original variable so that the new relationship is linear. For example, if the relationship between Y and X_1 is curvilinear, the relationship between log X_1 and log Y may be linear. Plots of all X's versus Y should be prepared to permit visual examination of the relationships. Also, the physical nature of the problem may suggest an underlying relationship between Y and X_i.
2. Find a nonlinear functional form through the use of polynomial regression. The form of the model is

$$Y = a_0 + a_1X + a_2X^2 + \ldots + a_nX^n \qquad (2)$$

The number of curves in the regression line depends on the degree of the polynomial (or highest exponent of X). The number of curves is always one less than the degree of the model.

3. Introduce interaction (or product) terms as additional variables.

The assumption in linear regression is that the effect of an X_i variable on Y is the same across all values of other X's. This means, for example, that the effect of asphalt surface thickness on the occurrence of alligator cracking is the same regardless of the level of other variables such as traffic, climate, or subgrade. If, however, the effect of asphalt surface thickness on the occurrence of alligator cracking is much different depending on whether the pavement is located in a warm or a cold climate, then an interaction exists between surface thickness and climate. The usual method of handling the problem of interaction is to use product terms of the two or more variables that interact (e.g., X_1X_2). Thus, a new variable X_1X_2 is created that is a function of both X_1 and X_2. The resulting equation is then

$$Y = a_0 + a_1X_1 + a_2X_2 + a_3X_1X_2 \qquad (3)$$

This model includes the additive effect of X_1 and X_2 and the interaction term X_1X_2, which represents the combined effect of X_1 and X_2 over and above the sum of a_1X_1 and a_2X_2. When more X's are involved, more interactive terms should be studied.

Boundary Conditions

The model should meet the boundary conditions that the physical situation requires. For example, when traffic-load-associated distress is being predicted, the distress prediction should be zero when no traffic loads have been applied. If PSI is predicted, the model should be capable of computing realistic values at the beginning and end of the pavement's life cycle.

Sensitivity Analysis

A sensitivity analysis shows the relative influence of changes in the independent variables on the dependent variable. The influence of the dependent variable (e.g., PSI or distress) can then be compared with experience or available data to see whether it is realistic. The sensitivity analysis can be conducted in various ways on the prediction model, but it should (a) be limited to the range of the variables used to derive the model and (b) consider the interrelationship of the variables. It should be noted that the least-squares regression coefficients are adjusted for other variables in the model. When the X variables are highly correlated, the individual coefficients do not provide an independent assessment of the effect of the variable on performance. Thus, it would be incorrect to vary only one variable (X) at a time to determine its influence on the performance (Y). A sensitivity analysis can be conducted in this case by varying all correlated variables in a reasonable way and noting their influence on performance.

The sensitivity analysis should be able to provide information to determine the general influence of the variables on performance, particularly the sign of the coefficient (+ or -); establish the relative importance of the variables; and determine deficiencies in the model.

STATISTICAL CRITERIA

Statistical Inference

Statistical criteria can be used to assess the precision of the estimated regression model. Initially, the form of the mathematical model must be assumed (such as Equation 1) based on the best judgment of the engineer. Then, after it is derived, it must be critically examined to either verify or reject the assumption. Two commonly used testing procedures are (a) the overall test for goodness of fit of the regression model and (b) the test for a specific regression coefficient. However, the residuals, as indicated by the standard error of the estimate, provide a direct indication of the precision of the model. It must be noted that, no matter how well the model fits the experimental data, if the data base is deficient, the model will also be deficient. Thus, the statistical tests relate only to the specific data used in the model's development. Another point is that the data base represents a sample of the population of all data, and thus the regression model developed from the data-base sample is only an estimate or inference of the true regression model that would be obtained if all possible data were used.

Error of Prediction

Whenever experimental data are used to develop a regression model, there will be a scatter of data about the line, as illustrated in Figure 3. The actual data points are given as (X_1, Y_1), (X_2, Y_2), etc. The X's represent the independent variable (e.g., slab thickness) and the Y's are the dependent variable (e.g., PSI or distress). A regression model of the form

$$Y = a_0 + a_1 X \tag{4}$$

is fitted to the data by using the least-squares technique. For a given value of X, say X_1, there is a difference between the actual value Y_1 and the predicted value $\hat{Y}_1$ obtained from Equation 2. This difference is called a residual or error in prediction: Residual = $Y_i - \hat{Y}_i$. The residuals contain all available information on the ways in which the regression model fails to explain the measured Y.

In developing a regression model from an in-service pavement data base, the variation about the regression line is caused by the following sources:

1. Test-equipment or observation-measurement repeatability errors (called testing errors),
2. Differences between the performance of supposedly identical pavement sections (called replicate errors), and
3. Model errors caused by the model having either an inadequate number of variables or an incorrect functional combination of variables.

Sources 1 and 2 of the total variation are called pure error, and source 3 is called lack-of-fit error. The pure error can be computed only if the data bank contains replicate projects having identical X values (e.g., structure, age, traffic, and climate). Any difference between replicate projects is assumed to be the result of random variation. Thus, when data collection is planned, it is important to obtain several replicate projects. Once the pure error is calculated, the degree of lack-of-fit error can be assessed, as shown in Figure 4.

The total residual sum of squares is computed as

$$SS_{res} = \Sigma(Y_i - \hat{Y}_i)^2 \tag{5}$$

It is composed of the pure-error sum of squares and the lack-of-fit sum of squares:

$$SS_{res} = SS_{pe} + SS_{lof} \tag{6}$$

If the mean squares attributable to pure error and lack of fit are computed, they can be compared, as shown in Figure 4. If they are significantly different, then the proposed regression model suffers from lack of fit and is inadequate, and a new model should be developed. If not, there is no reason to doubt the adequacy of the regression model (based on available data).

The standard error of the estimate (SEE) is computed as

$$SEE = \sqrt{\Sigma(Y_i - \hat{Y}_i)^2/(N - 2)} \tag{7}$$

The magnitude of the SEE is simply the standard deviation of the residuals and may be interpreted approximately as the "average error in predicting Y from the regression model" (12). The size of the SEE can be compared with the mean of all Y's. Ideally, the SEE should be much smaller than the mean of all Y's.

The residuals $(Y_i - \hat{Y}_i)$ should be carefully examined. Various plots can be prepared, such as (a) overall plot of residuals, (b) time sequence if order is known or meaningful, (c) plot against the calculated $\hat{Y}$ values, (d) plot against the independent X variables, and (e) any way that makes sense for the model under evaluation (11).

Multiple Correlation Coefficient (R^2)

The overall accuracy of the regression model can be assessed by the multiple correlation, R^2. The total sum of squares in Y (which represents the overall variation of the performance variable Y) consists of two parts:

$$SS_y = SS_{reg} + SS_{res} \tag{8}$$

where

$SS_y = \Sigma(Y - \bar{Y})^2$,
$SS_{reg} = \Sigma(\hat{Y} - \bar{Y})^2$, and
$SS_{res} = \Sigma(Y - \hat{Y})^2$.

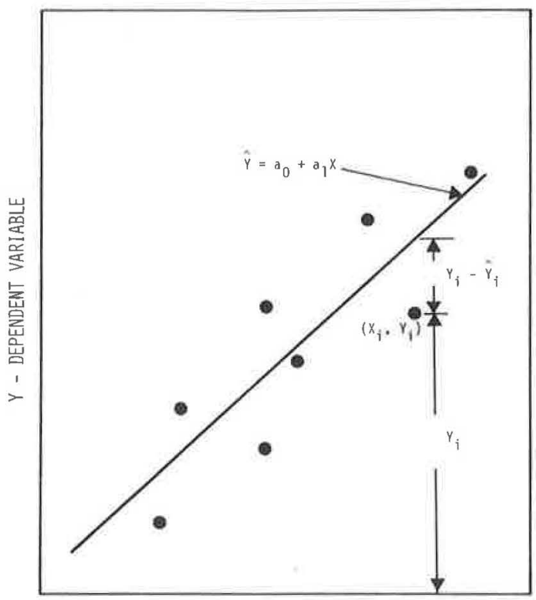
If these sources of variations are used, a natural measure of prediction accuracy is obtained, as follows:

$$R^2 = SS_y - SS_{res}/SS_y = SS_{reg}/SS_y \tag{9}$$

Thus, the R^2 represents the proportion of the total variation in Y explained by the regression model. The closer that R^2 is to 1.0, the closer the data cases lie on the predicted line, and the closer to 0.0, the greater the scatter of data about the line.

A statistical test can be employed to test the null hypothesis that the R^2 is zero. This would indicate that there is no significant relationship between any of the X variables and the Y performance variable. It is equivalent to the null hypothesis that all a_i regression coefficients are equal to zero. Thus, if the null hypothesis is rejected, one or more of the coefficients has an absolute value greater than zero. The test does not determine which a_i value is nonzero. Thus, further tests are made on the regression coefficients, as explained later. The value of R^2 that is statistically significant at some level of significance varies greatly as the number of data cases used to develop the equation changes. The larger the data bank, the lower the R^2 needed to reject the null hypothesis (and thus accept the alternative hypothesis that correlation exists). Regression analysis with in-service pavements typically produces relatively low R^2 values because of the large variations and many unknown factors that affect performance.

Figure 3. Illustration of typical scatter of data about a linear regression line.



Regression Coefficients

The regression coefficients (a_i) represent the expected change in Y with a change in one unit in one X variable when all other X's are held constant. In the stepwise regression technique, the coefficient of each variable is tested as it enters the equation to see whether it differs significantly from zero. This test can be used (and normally is) as a criterion to decide whether or not to enter a given variable into the regression model. If the null hypothesis of $a_i = 0$ cannot be rejected, then it can be concluded that the X variable does not significantly affect the Y variable, and it should not be allowed in the regression model.

In summary, the model should meet the following statistical criteria:

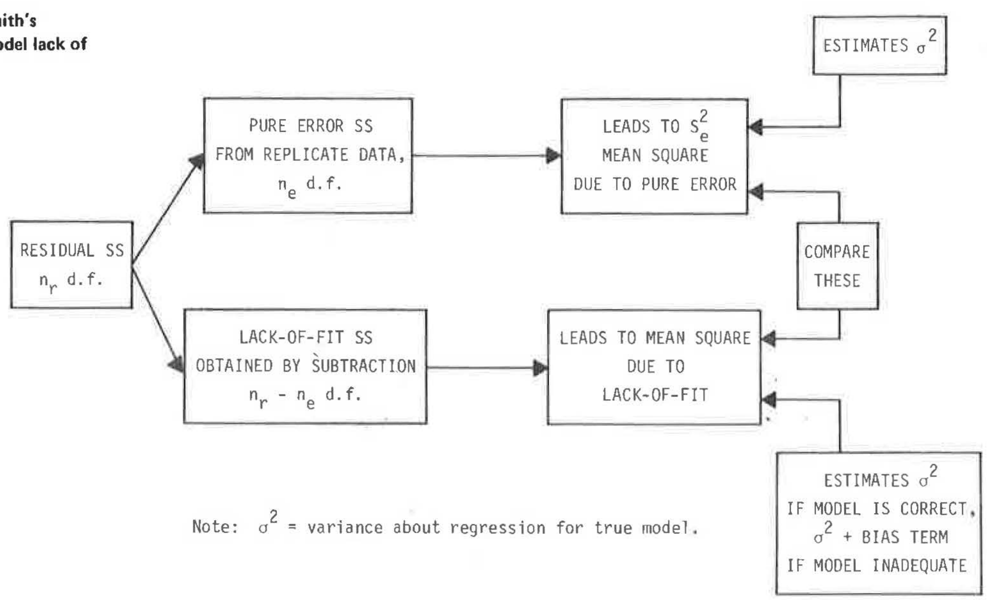
1. The final model should explain a high percentage of the total variation about regression (or R^2) to indicate the overall model accuracy;
2. The standard error of the estimate of the model should be less than a specified practical value (e.g., < 0.20 of the mean);
3. The model should not suffer from significant lack of fit;
4. All estimated coefficients of the X variables should be statistically significant with, say, $\alpha \leq 0.05$; and
5. There should be no discernible patterns in the residuals.

CONCLUSIONS

The development of and requirements for reliable models for the prediction of pavement peformance are presented.

1. An adequate data base from which to build the model is most important. Adequacy is defined in terms of having a representative unbiased sample of projects, reliable and accurate data, and a sufficient number of data cases that include some replicates.
2. The model must include all variables that have significant influence on the performance. If

Figure 4. Draper and Smith's determination (11) of model lack of fit.



Note: σ^2 = variance about regression for true model.

it does not, the limitations of the models should be identified. Mechanistic variables such as stress, strain, or stress-and-strength ratio should be considered, since this may greatly increase the reliability of the model. The stepwise regression procedure is believed to give the best selection of independent variables.

3. The functional form of the model should be carefully selected to represent the physical real-world situation as closely as possible. This will lead to a model that considers the appropriate shape, nonlinearity, and interactions of variables; meets boundary conditions; and also gives reasonable sensitivity of the variables. Such selection requires extensive knowledge of the problem and the available data.

4. Various statistical criteria should be used to assess the precision of the model. The model should explain a high percentage of the total variation about regression; the standard error should be less than a practical value for usefulness; there should be no discernible patterns in the residuals; the model should not suffer from significant lack of fit; and all estimated coefficients should be significant with, say, $\alpha \leq 0.05$. Detailed explanation of regression-model development and testing may be found in the literature (11-15).

5. Significant progress can be made in pavement technology if agencies will begin the development of in-service pavement data bases from which reliable predictive models can be developed and used for pavement management purposes.

ACKNOWLEDGMENT

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Characterization of Bitumen-Treated Sand for Desert Road Construction

GURDEV SINGH AND SHAFIQ KHALIL HAMDANI

This paper summarizes the findings of an experimental program designed to permit evaluation of the accumulation of permanent deformation in bitumen-treated sand layers by applying the more rational methods of pavement analysis and design. The primary part of the work consists of the characterization of the cumulative deformation response of bitumen-sand specimens tested under simulated conditions of temperature and dynamic stress. Results have been analyzed by using multiple regression analysis, and predictive relationships of rut depth are formulated therefrom.

The developing economies of many countries in the Middle East have resulted in an increasing demand

Table 1. Growth in number of motor vehicles in some Middle Eastern countries.

		Units (000s)		
Country	Code	1966	1970	1975
Algeria	A	103.4	142.8	180.0[a]
	B	68.3	81.6	95.0[a]
Egypt	A	105.3	130.7	215.5
	B	27.5	30.1	46.3
Iran	A	142.4	278.2	589.2[a]
	B	49.4	73.5	111.2[a]
Kuwait	A	69.6	112.9	203.7
	B	25.3	36.8	68.5
Lebanon	A	105.4	136.0	220.2[a]
	B	14.1	16.6	23.4[a]
Libyan Arab Republic	A	53.0	100.1	263.1
	B	25.2	45.4	131.3
Morocco	A	168.6	222.5	320.1
	B	68.0	83.9	127.2
Tunisia	A	53.7	66.4	102.6
	B	31.5	37.2	67.0

Note: A = passenger cars; B = commercial vehicles.
[a]Statistic for 1974.

for the transportation of passengers and goods in general. This increasing demand for road transport is exerting an urgent need for the construction of many miles of both high-standard surfaced roads and secondary, low- to medium-volume desert roads; see Table 1 (1).

Most countries in this region either border or lie completely within areas whose climate is categorized as desert or semidesert, and a significant proportion of the total required mileage is projected to run through desert land where little or no coarse hard aggregates or gravel are readily available for conventional road construction. The surface soil consists largely of only windblown or other sand deposits. The problem of how to build such roads economically is therefore becoming evident.

One of the important items influencing the economy of road projects is the cost of aggregates. Aggregates constitute 95-100 percent of the material in the base layers and subbases (2), and the availability, or lack, of economically exploitable sources of suitable aggregate has a great impact on the total cost of road projects.

Stabilization of windblown and other sand deposits by using bituminous products has been tried in many countries all over the world, and the literature reveals many successful experimental field trials that emphasize the potential of this composite in road construction (3-5). The inherent lack of internal stability of these mixes, however, coupled with excessive temperature susceptibility, makes surface rutting a major cause of failure in desert roads. Unfortunately, the design subsystem of this type of damage has received little attention compared with fatigue and thermal-cracking failure (6). In addition, more rational methods of design against rutting (such as pseudoelastic and linear viscoelastic designs) have been rendered inapplicable by the lack of suitable characterization of the mechanical response of the materials considered in the design. Accordingly, less reliable empirical methods for mixture and thickness design have been and are still being widely used in the construction of bitumen-sand roads.

In this work, the systems approach has been adopted to investigate the potential of using bitumen-treated, poorly graded sand (like that encountered in real desert-road projects) as a structural component in pavement systems. The approach consists of simulative tests in which a temperature-controlled triaxial cell capable of applying repetitive and independent stresses in the axial and the radial directions was used to determine the pattern in which the irrecoverable component of the dynamic strain accumulates under different testing conditions.

METHODOLOGY

A general outline of research activities is shown in Figure 1. The primary part is represented by cells 11 to 15; other cells represent the necessary activities of preparatory or complementary nature.

EXPERIMENTAL PROGRAMS

The work involved the design and implementation of two types of experimental programs. The first consists of four preparatory investigations intended to provide necessary data for a detailed design of the two primary studies.

Preparatory Investigations

1. Determination of the compactibility or the compaction-density relationships for a range of bitumen-sand mixtures--The gyratory testing machine (GTM) was used to apply two different compactive efforts to 11 mixes that ranged in bitumen content from 2 percent to 13 percent (in 1 percent increments). The test program was replicated according to the 1975 ASTM standard D3387-74T so that each treatment was represented by three tests. Unit weights corresponding to the lower compactive effort (7) were adopted as molding densities for the preparation of the specimens for the subsequent study of mixture design.

2. Mixture-design study to determine the optimum amount of bitumen to be mixed with the sand--The cylindrical specimens tested were 51 mm in diameter and 102 mm long. These were compacted in a constant-volume mold (British Standard 1924: 1975) to initial densities corresponding to bitumen contents of 3, 5, 7, 9, and 11 percent, as determined above. The temperature-controlled dynamic triaxial apparatus was used to apply repeated uniaxial stresses, and the criterion adopted was the resistance of the tested samples to the cumulative permanent distortion.

3. Determination by direct testing of the resilient modulus and Poisson's ratio of the optimum mixture chosen--At this stage, measurements of variations in these parameters with stress repetitions were considered unnecessary. The measurement of the dynamic response at each testing condition was, therefore, made only once and then only after a conditioning stage of 100 repetitions. A range of temperature and stress conditions likely to exist in real pavements was included in the test program. The duration of the repeated stress pulses was varied at two levels, 0.1 s and 0.2 s. If an element is considered to be 200 mm beneath pavement surface, these durations correspond approximately to wheel speeds of 26 km/h and 16 km/h, respectively (8). Stress amplitudes, on the other hand, were set to result in ratios of σ_3/σ_1 between 0.0 and 0.5, thus simulating only those combinations of stress that occur in the compression zone of a pavement layer. Stress-strain response was recorded on an ultraviolet (UV) trace that was subsequently used to calculate the resilient constants.

4. Structural analysis of typical desert-road cross sections to define the stress profiles and combinations likely to exist in compacted layers of

the optimum mixture selected--The program consisted of analyzing 27 systems in which the treated sand acted either as the top layer or as a base under 100 mm of bituminous concrete surfacing. The subgrade in all the systems was assumed to be untreated and densely compacted sand. Resilient moduli and Poisson's ratios for the bitumen-sand layer were chosen from results of the investigation described above, and stiffness moduli for the bituminous concrete surfacing were selected by using van der Poel's nomograph (9), assuming certain temperature extremes (10). Values for the resilient modulus of the dense sand subgrade were derived from two sources: (a) experimental results reported by other investigators (11,12) and (b) by measuring the California bearing ratio (CBR) and using it in the correlation: E_{dyn} (kN/m²) = (5 to 20)10^3 x CBR(%), suggested by Huekelom and Foster (13).

The multilayer elastic analysis was facilitated by using Shell's computer program, Bitumen Structures Analysis in Roads (BISAR).

Primary Investigation

The selection of constitutive equations that adequately model the response of paving materials to the loading and environmental conditions can be a very complex task. Simulation is a suitable technique for solving such complex problems. It is,

Figure 1. Outline of research activities.

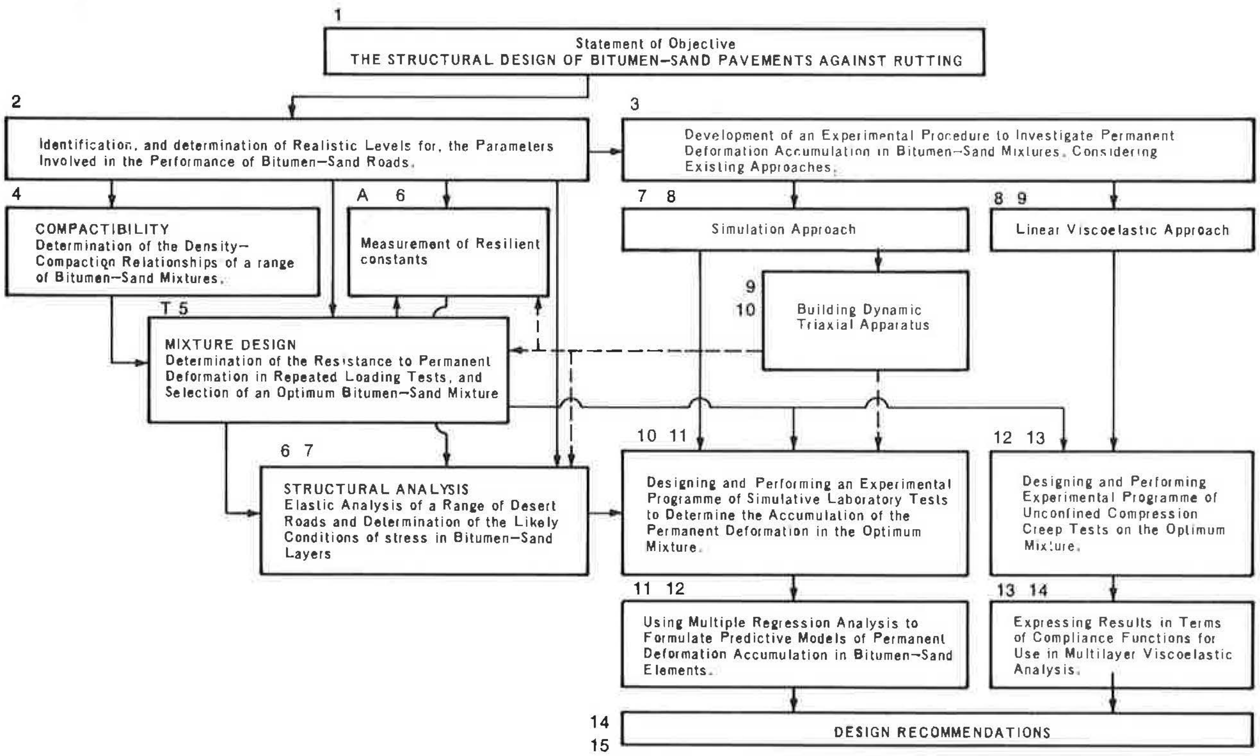
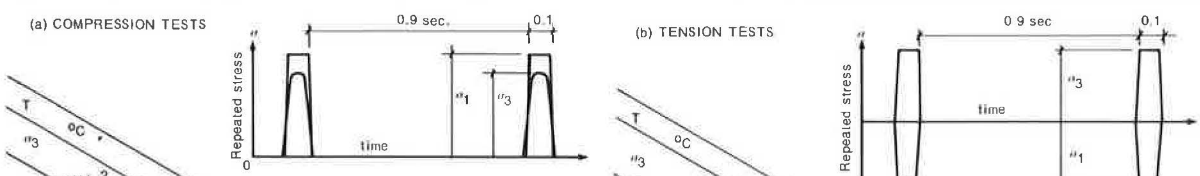


Figure 2. Experimental program for the simulation of the permanent deformation accumulation in bitumen-sand layers under repetitive triaxial stresses.



(a) COMPRESSION TESTS

σ_1 kN/m²	σ_3 kN/m²	T °C: 25	32.5	40
276	138	4	13	3
	483	2	14	1
362	138	9	17	11
	483	10	18	12
448	138	8	15	7
	483	6	16	5

(b) TENSION TESTS

σ_1 kN/m²	σ_3 kN/m²	T °C: 10	20	30
−27.6	13.8	22	31	21
	69	20	32	19
−55.2	13.8	27	35	29
	69	28	36	30
−82.8	13.8	26	33	25
	69	24	34	23

NOTES σ_1 & σ_3 axial and radial repetitive stresses

Figures in cells are to identify the corresponding treatments.

Mixture tested : optimum bitumen-sand (7% B.C.)

Each treatment is represented by 4 tests.

Figure 3. Specified gradation of the sand used and typical gradations in desert areas.

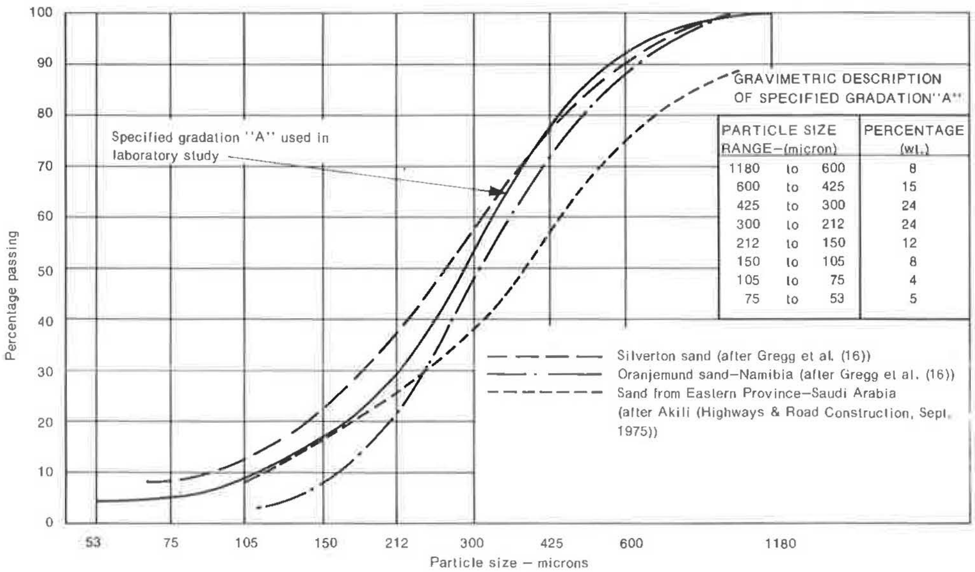


Figure 4. General schematic diagram of equipment.

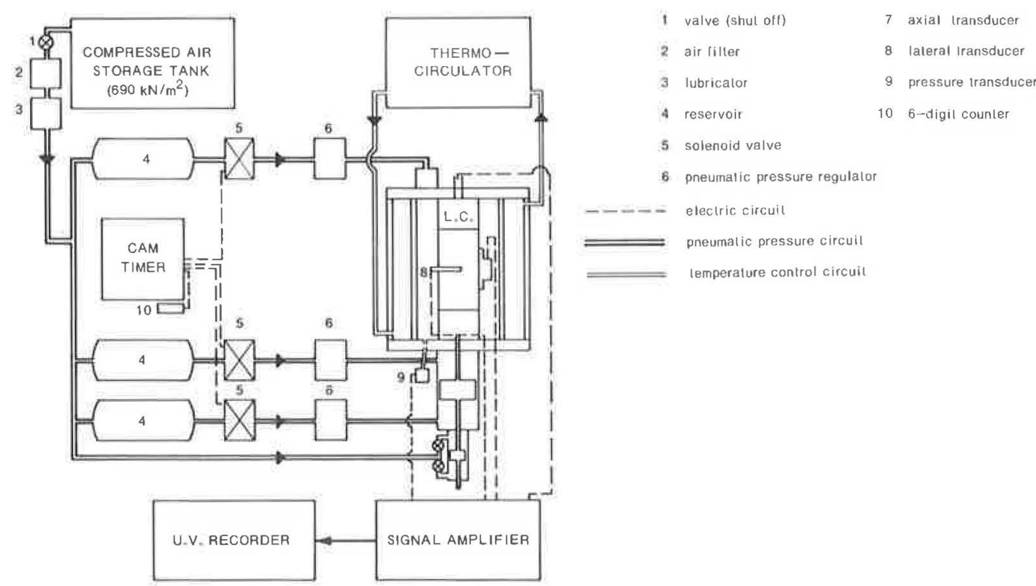
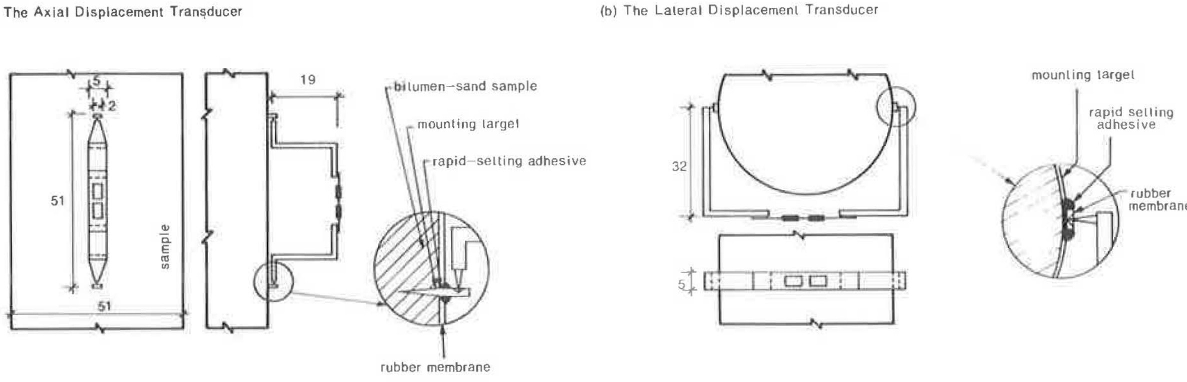


Figure 5. Transducers for load and displacement measurements in the dynamic triaxial apparatus.



however, not necessary to include most of the actual features of the problem considered, since few are vital to the particular aspect under study (14). The extremely arid environment of hot desert implied that moisture content of samples could be excluded as a test variable. Also reduced to constants are factors of gradation and surface characteristics of the sand. Bulk density and bitumen content of tested samples were maintained at 1857 kg/m^3 and 7 percent, respectively.

The experimental program is shown in Figure 2. Amplitudes for vertical and horizontal stress were selected with consideration of actual stress profiles in bitumen-treated sand layers produced in the previous investigation of stress analysis and with a provision that stresses should be repeated at least 100 000 times in the compression tests and 30 000 times in the tension tests before permanent distortion in the tested sample should reach 10 percent extension or contraction. Stresses in the tension tests were also kept relatively low to limit the number of samples that tended to fracture prematurely.

MATERIALS AND EQUIPMENT

Materials

The materials procured for preparing test specimens consisted of poorly graded quartzitic sand from Leighton Buzzard (United Kingdom), 70/80-penetration-grade bitumen, and solvent naphtha. The sand was the only aggregate used in preparing the mixtures. Its type and gradation were selected to approximate those of windblown sand often encountered in desert-road construction. Figure 3 shows typical gradation curves for sands of this type (15), together with the distribution curve adopted for this work. The shape of the particles was classified as rounded to irregular and the surface texture as smooth. The percentage of fines was fixed at 5 percent. Possible variability because of unspecified gradation of this proportion of fines was eliminated by sieving out all dust particles smaller than 53 µm.

A rapid-curing cutback was prepared by diluting the refinery bitumen by 40 percent of its volume by solvent naphtha. The proportion of solvent was arrived at after a few preliminary trials to allow

Figure 6. Compactibility of bitumen-sand mixtures in the gyratory testing machine.

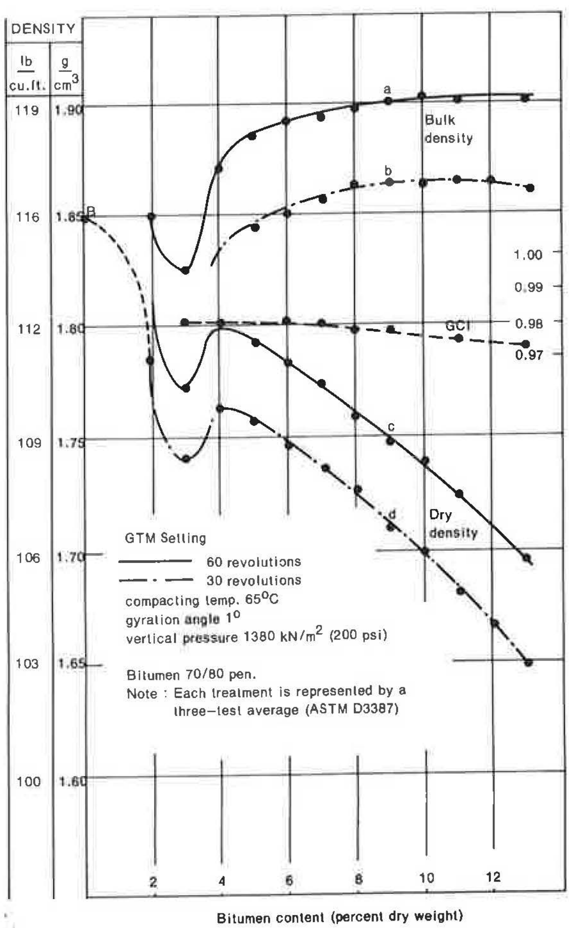
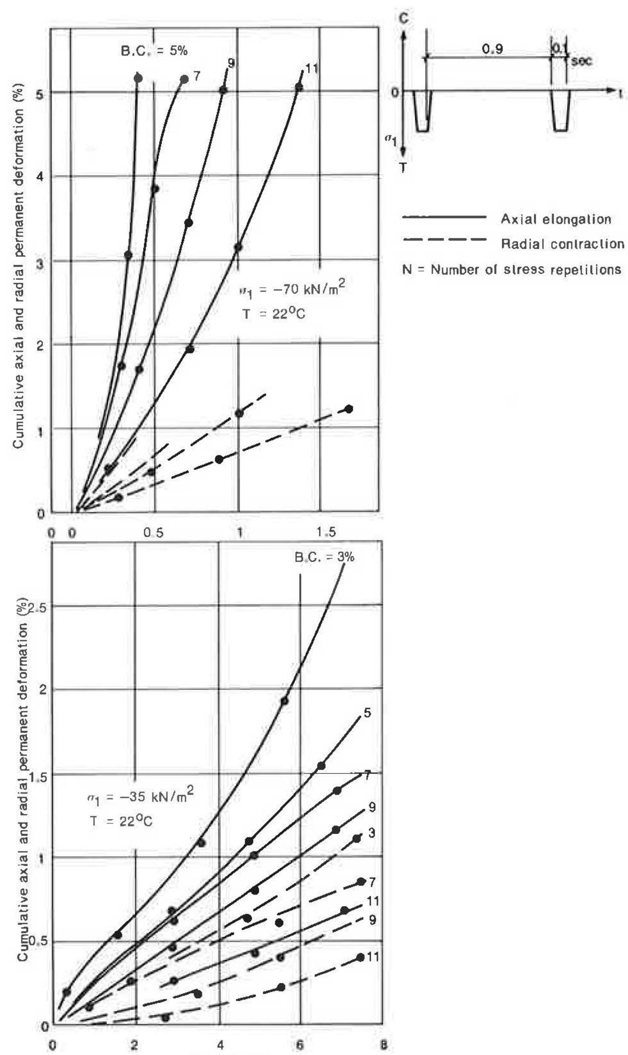


Figure 7. Accumulation of permanent deformation in bitumen-sand mixtures under repeated uniaxial stress–tension.



adequate manual mixing with the minimum of preheating.

Equipment

The triaxial cell fabricated and used here is capable of controlling the temperature and of pulsing the axial and radial stresses independently. A Perspex cylinder was placed around the pressure chamber to form an annular cavity through which water, at the predetermined temperature, was circulated. The pressure chamber was filled with a relatively incompressible fluid that served as a heating and a lateral-pressurizing medium. Amplitudes of axial and radial pressure pulses were controlled by pneumatic pressure regulators connected to the air-supply line. Durations and synchronization of pulses were ensured by using three solenoid valves connected to the pressure lines and controlled by an electric cam timer. A schematic diagram of the equipment is shown in Figure 4.

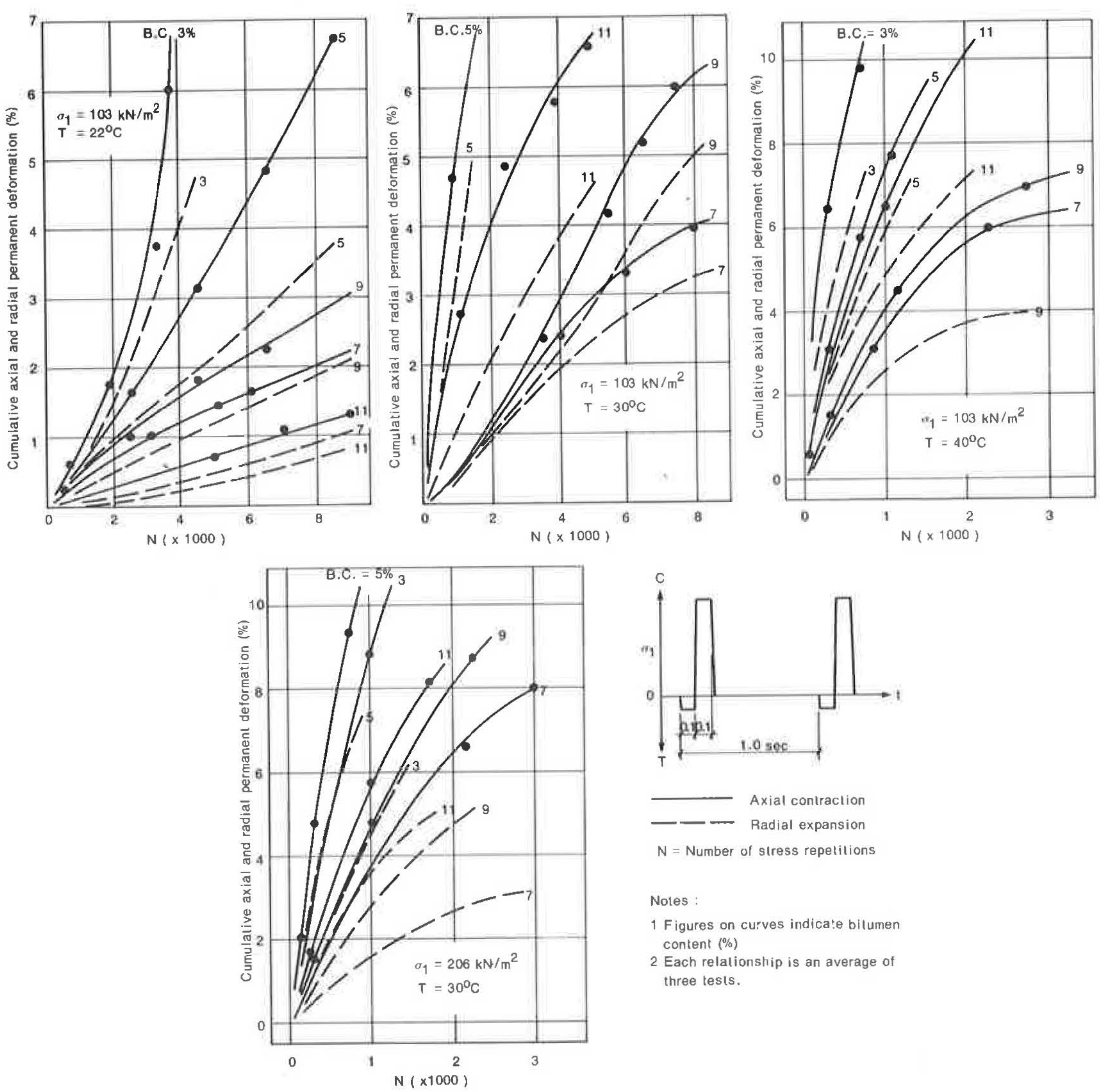
The resilient and the cumulative permanent deformation in the sample were measured by using lightweight transducers made of rigid aluminum arms connected to central pieces of thin brass plates, as shown in Figure 5. Strain gauges were bonded to these plates to form full-bridge circuits.

Axial stress was measured inside the pressure chamber by using a load transducer bearing directly on the specimen. It was manufactured so that the measurement was independent of chamber pressure (16). Finally, confining pressure was measured by using an ordinary commercial transducer.

RESULTS AND DISCUSSION

Results of the GTM compaction are plotted in Figure 6 as bitumen content versus density relationships. It can be seen that, for most of the range, dry density decreases markedly whereas bulk density increases slightly with binder content. This indicates that the reduction in mix density resulting from particle separation caused by the addition of bitumen is a little more than compensated for by the weight of the added binder. In the kneading-compaction process, the high porosity of sand seems to allow a larger proportion of the added binder to occupy existing voids rather than to force particles apart. This results in the absence of a definite optimum amount of bitumen for

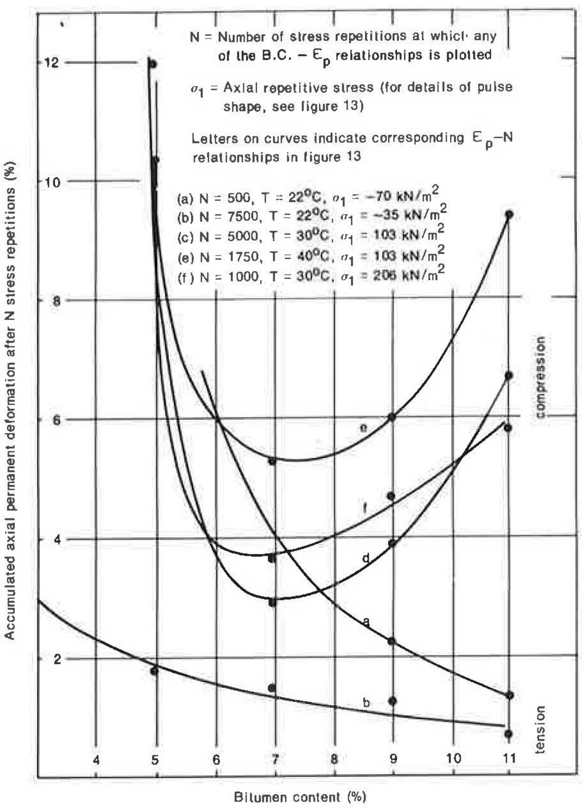
Figure 8. Accumulation of permanent deformation in bitumen-sand mixtures under repeated uniaxial stress—compression.



maximum bulk density, which is regarded as a characteristic property of the compactibility of the mixes used.

Results of the experimental program of mixture design are presented in Figures 7, 8, and 9. It is obvious that, in general, repeated tension tests resulted in an increasing rate of permanent deformation, whereas compression tests produced a decreasing rate with stress repetition, probably because of the slight densification and increase in sample stiffness effected in the compression tests compared with the weakening of tension samples caused by initiation of microcracks in the fatigued films of binder (6). Comparison of the five mixes in Figure 9 shows the resulting percentages of permanent distortion caused by the same number of stress cycles. The effect of bitumen content on the resistance to cumulative distortion appears to differ in the tension and compression tests: Although in repeated tension the resistance continuously increased, it is seen to reach a maximum at about 6.5-7.5 percent bitumen content in most of the repeated compression tests. The optimum amount of bitumen in these tests corresponds, therefore, to the critical mix condition at which the highest mobilization of the combination of cohesion and friction occurs. The results thus seem to point to the use of richer mixes in the lower portions and about 7 percent binder in the upper portions of pavement layers for an optimum performance against rutting.

Figure 9. Bitumen content versus permanent deformation accumulation in the design of bitumen-sand mixtures.



Experimental values of the resilient modulus M_r and Poisson's ratio ν_r for the selected design mix (7 percent bitumen content and 1857 kg/m^3 bulk density) are shown in Figure 10. Although the effect of temperature on these values is marked, the effects of stress amplitude and duration appear to have inconsistent trends, as indicated by the shaded areas. These figures were used to select realistic values of elastic constants that corresponded to extreme layer conditions in order to determine the stress profiles.

It was evident from the relative values of layer moduli in the desert pavements analyzed that the general stress patterns in these systems would differ significantly from those in conventional cross sections. In the latter type, the ratio of the base to subgrade stiffness can be as high as 20 or more. For a desert road that consists of a bituminized sand layer on dense and untreated sand subgrade, this ratio would be only about 4. This ratio occurs when the binder is 70/80 penetration grade, the treated sand layer is relatively cold, and the subgrade is at its weakest condition.

Results of the stress analysis program are shown in Figures 11 and 12. These show the range of variation in the bitumen-sand layer of the vertical and horizontal stresses with depth and with consideration of possible extremes in the properties of the layers under real service conditions. It is clear that the influence of these properties becomes less pronounced when the thickness of the treated-sand layer is increased. The reduction in

Figure 10. Resilient parameters of the optimum bitumen-sand mixture.

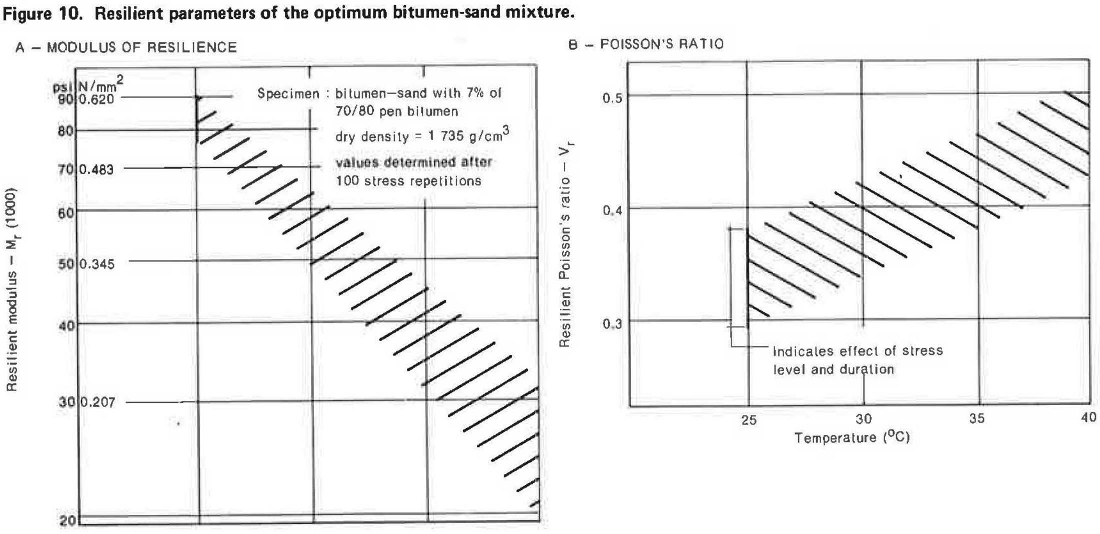


Figure 11. Range of stresses in the bituminized sand layer of two-layer pavement systems.

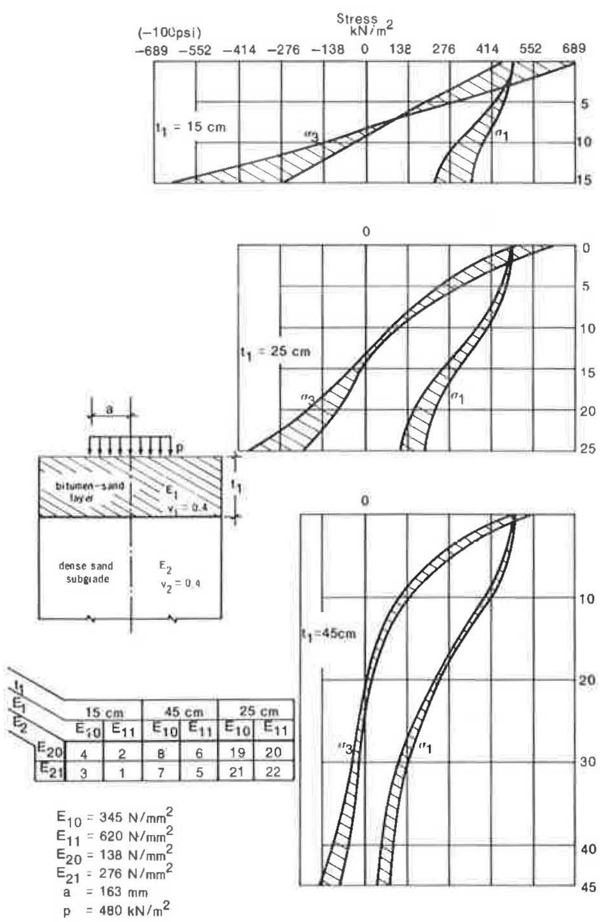
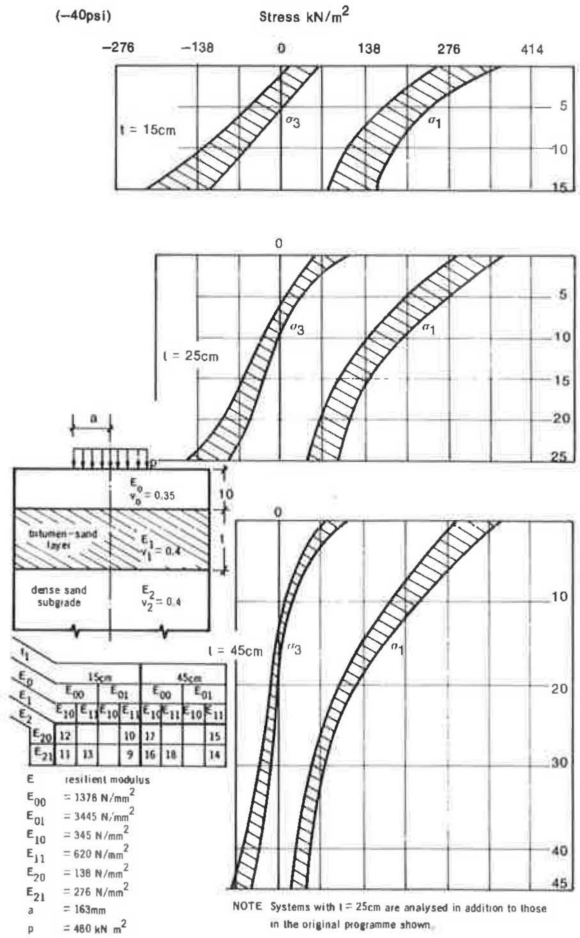


Figure 12. Range of stresses in the bituminized sand layer of three-layer pavement systems.



the horizontal tensile stress range at the bottom amounts to about 80 percent for an increase in thickness from 150 to 450 mm (Figure 11). Corresponding decrease in the vertical compressive stress at the same location is around 67 percent.

Figure 13 shows the relationships corresponding to all treatments of the program on one semilogarithmic plot that proved useful in further analysis. Although they are nonlinear, a significant portion of the relationships in this figure can be approximated by straight-line segments, i.e., where N is greater than 1000. The least-squares technique was, therefore, applied to perform such an approximation by using the original four-test data in each treatment. In Figure 14, the resulting straight-line equations are listed in terms of the slope $S_c = \varepsilon_\rho/\log N$ and the intercept with N = 1000 ordinate, I_c. The formulation of equations for predicting these two dependent variables, when temperature and stress conditions are given, would allow the estimation of the amount of distortion accumulating in an element of any thickness.

Such equations were derived here by using multilinear regression analysis of the experimental data in Figure 14. The slope (S_c) or the intercept (I_c) represented the dependent variable, while factors of temperature (T) and stress (σ_1 and σ_3) were input, in different combinations, as independent variables. The combination terms, which represented potential interaction effects, included such terms as $(\sigma_1-\sigma_3)$, $(\sigma_1-\sigma_3)T$, $\sigma_1 T$, $\sigma_3 T$, $\sigma_1\sigma_3$, and others. The facility of stepwise multiple regression provided by the Statistical Package for the Social Sciences (17) allowed the choice of the fewest terms with the highest predictive power of the dependent variable. The equations finally selected are included in Figure 15, together with their characterizing statistics. The figure also includes selected equations for predicting S_t and I_t associated with the tension-test results. These were derived in the same way as S_c and I_c but from an ordinary plot of ε_ρ versus N, as shown in Figures 16 and 17.

In any of the four equations selected, the term for the interaction between deviatoric stress and temperature proved sufficient to predict the dependent variable to an acceptable level. Values of the squared multiple correlation coefficient (R^2) indicate that at least 82 percent of the variance in S or I is being accounted for by the equations selected. This reflects a simplicity in the distortion response that could be related to the very nature of the mixture used. Unlike well-graded aggregates in conventional mixes, the rounded shape and smooth texture of the stabilized sand particles produced a minimum degree of interlocking and, consequently, the least interruption to the function of the binder film in between. Minimum interlocking is also encouraged by the absence of large-sized aggregates, which implied that load transmitted between any two adjacent particles would be relatively very small. It is the presence of the

Figure 13. Results of the experimental program of repeated-load triaxial tests (compression series).

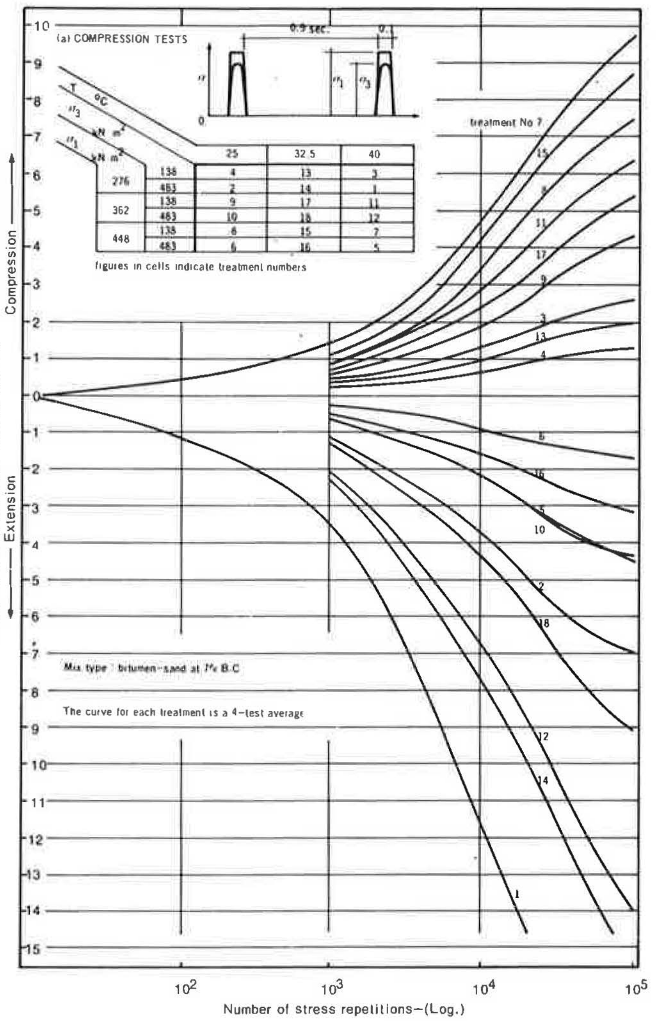
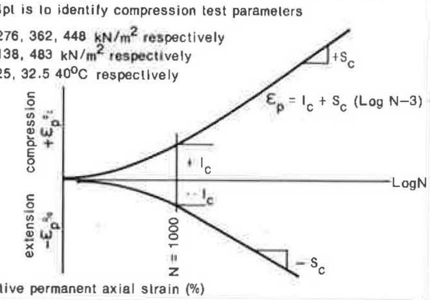


Figure 14. Characteristics of the relationships between ϵ_p and log N approximated to linearity (compression tests: N > 1000).

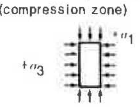
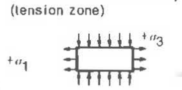
TREATMENT No	REPEATED STRESS AMPLITUDE: AXIAL σ_1	REPEATED STRESS AMPLITUDE: RADIAL σ_3	TEMPERATURE °C	I_c* INTERCEPT WITH N = 1000 ORDINATE (ϵ_p %)	S_c GRADIENT $\frac{\epsilon_p\%}{\text{Log N}}$
1	σ_1'	σ_3''	T'''	−3.16	−8.45
2	σ_1'	σ_3''	T'	−0.95	−3.06
3	σ_1'	σ_3'	T'''	0.30	1.12
4	σ_1'	σ_3'	T'	0.15	0.57
5	σ_1'''	σ_3''	T'''	−0.54	−1.99
6	σ_1'''	σ_3''	T'	−0.18	−0.77
7	σ_1'''	σ_3'	T'''	1.05	4.16
8	σ_1'''	σ_3'	T'	0.42	3.53
9	σ_1''	σ_3'	T'	0.27	1.96
10	σ_1''	σ_3''	T'	−0.49	−1.95
11	σ_1''	σ_3'	T'''	0.51	2.86
12	σ_1''	σ_3''	T'''	−1.49	−5.97
13	σ_1'	σ_3'	T''	0.22	0.88
14	σ_1'	σ_3''	T''	−1.68	−6.78
15	σ_1'''	σ_3'	T''	0.64	3.99
16	σ_1'''	σ_3''	T''	−0.36	−1.37
17	σ_1''	σ_3'	T''	0.36	2.42
18	σ_1''	σ_3''	T''	−0.95	−3.91

* The c subscript is to identify compression test parameters
$\sigma_1', \sigma_1'', \sigma_1'''$ = 276, 362, 448 kN/m^2 respectively
σ', σ'' = 138, 483 kN/m^2 respectively
T', T'', T''' = 25, 32.5 40°C respectively



ϵ_p% = cumulative permanent axial strain (%)

Figure 15. Selected predictive equations and their characteristics.

Repeated Stress Pattern (in the Triaxial Test)	Predictive Equation Selected	Characteristics: STD. ERROR B	BETA	STANDARD ERROR	R^2	F
Triaxial Compression (compression zone) 	$I_c = -0.57438 + 0.00015\,(\sigma_1 - \sigma_3)\,T$	0.00002	0.91	0.4427	0.82	75
	$S_c = -1.67194 + 0.00057\,(\sigma_1 - \sigma_3)\,T$	0.00004	0.96	1.0813	0.92	193
Biaxial Compression σ_3, with Uniaxial Tension σ_1 (tension zone) 	$I_t = 0.21447 + 0.00072\,(\sigma_1 + \sigma_3)\,T$	0.00006	0.94	0.2882	0.89	135
	$S_t = -1.45665 + 0.00250\,(\sigma_1 + \sigma_3)\,T$	0.00014	0.97	0.6758	0.94	300

Notes – In the above equations, temperature T, and stress σ_1 and σ_3, should be in °C and kN/m^2 units.
– Signs of the resulting parameters I or S are significant for I_c and S_c a plus sign indicates shortening, a minus sign indicates elongation. For I_t and S_t, a positive sign should normally be expected, indicating permanent compression.

Figure 16. Results of the experimental program of repeated-load triaxial tests (tension series).

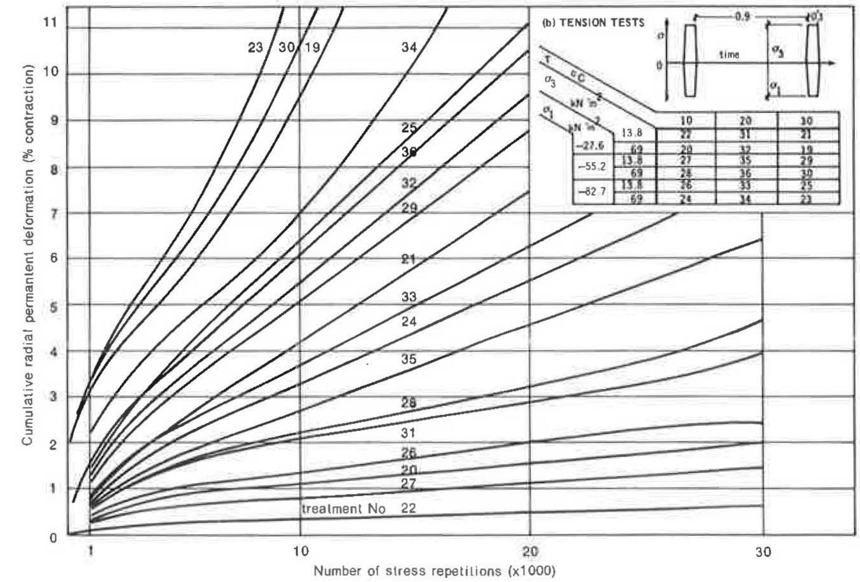


Figure 17. Characteristics of the relationships between ϵ_p and N approximated to linearity (tension tests: N > 1000).

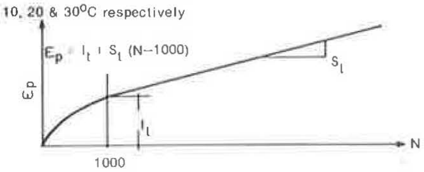
TREATMENT No	REPEATED STRESS AMPLITUDE AXIAL σ_1	RADIAL σ_3	TEMPERATURE °C	I_t* INTERCEPT WITH N=1000 ORDINATE (ϵ_p%)	S_t GRADIENT $\frac{\epsilon_p\%}{N}$ (10^{-4})
19	σ_1'	σ_3''	T'''	2.93	6.9
20	σ_1'	σ_3''	T'	0.69	0.4
21	σ_1'	σ_3'	T'''	1.47	3.0
22	σ_1'	σ_3'	T'	0.23	0.1
23	σ_1'''	σ_3''	T'''	3.07	10.3
24	σ_1'''	σ_3''	T'	1.17	2.3
25	σ_1'''	σ_3'	T'''	2.12	4 7
26	σ_1'''	σ_3'	T'	0.85	0.5
27	σ_1''	σ_3'	T'	0 52	0.3
28	σ_1''	σ_3''	T'	1 08	1 2
29	σ_1''	σ_3'	T'''	1 80	3 7
30	σ_1''	σ_3''	T'''	3 08	8 3
31	σ_1'	σ_3'	T''	1 18	0.9
32	σ_1'	σ_3''	T''	1 87	4.0
33	σ_1'''	σ_3'	T''	1.28	2.7
34	σ_1'''	σ_3''	T''	2 26	5.4
35	σ_1''	σ_3'	T''	1 09	1 8
36	σ_1''	σ_3''	T''	2 09	4 4

* The t subscript is to identify tension test parameters

$\sigma_1', \sigma_1'', \sigma_1'''$ = 27.6, 55 & 83 kN/m^2 respectively (tension)

σ_3', σ_3'' = 13.8 & 69 kN/m^2 respectively

T', T'', T''' = 10, 20 & 30°C respectively



ϵ_p = cumulative permanent radial strain (contraction)

aggregate phase and the complexity of its structure that contribute to the complexity of the mixture (18,19).

The relatively large sample size adopted in the experimental phase (four specimens per treatment) has helped in producing more accurate estimates of the real response. It limited the variance associated with the average response in each treatment, which had the effect of reducing inconsistencies in the input data to the regression analysis. [It is interesting to note that Shen and Smith (20), in their work on chemically stabilized fine sand, observed a simplicity of response similar to the one reported here.]

The plausibility of the formulated predictive equations (Figure 15) was finally demonstrated by a numerical example. Here the equations were used to estimate the component of surface-rut depth caused in the bitumen-sand layer of two desert pavement cross sections. Certain assumptions regarding traffic and material variables were made, and the geometries of the sections were chosen to allow observation of the effects of changing the thickness of the layer on the value of the resulting rut component at relatively high average temperature. Computation and other details are included in Figures 18 and 19. It is useful to note that because of the lack of complete simulation in the tests of the mode of tensile stresses in the pavement, as well as because of the assumptions written into the BISAR program, the results reported here are overestimates of the total permanent strain in the tensile zone.

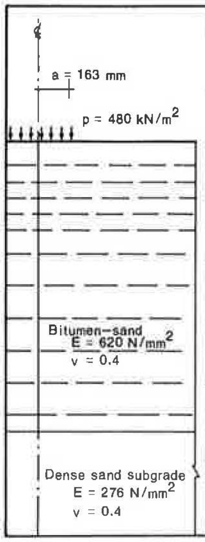
CONCLUSIONS

1. For a type of sand similar in properties to that encountered in desert road construction, design bitumen content for optimum performance under repeated uniaxial compressive stresses lies between 6 percent and 8 percent; under repeated uniaxial tension, no optimum value exists within the range of testing conditions adopted.

2. Stress pulse duration between 0.1 s and 0.2 s has no clear trend in its effect on the resilient modulus of the optimum mix in the triaxial mode of testing.

3. The permanent deformation response of the optimum mix under repeated triaxial stresses was investigated, the results were analyzed by multiple linear regression, and predictive relationships for distortion accumulation were formulated. The response correlated highly significantly with the term $(\sigma_1-\sigma_3)T$, which represents the interactive effect among the main factors of axial stress σ_1, lateral stress σ_3, and T. This term

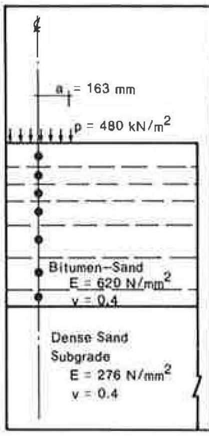
Figure 18. Example of prediction of permanent deformation in a 45-cm bitumen-sand base.



Repeated Stresses (kN/m²)		T	N	Solutions Using Equations In Table 3				Results Using Equations In Table 3			
				ε_p at N = 1000		ε_p rate after N = 1000		Total Vertical Permanent Deformation In The Element At N = 7000			
				I_c	I_t	S_c	S_t	ε_{pc}		ε_{pt}	
σ_1	σ_3			%	%	%/Log N	$\frac{\%}{N}$ (10^{-4})	%	mm	%	mm
479	358	25 °C	7000	−0.121		0.052		−0.07	−0.03		
469	260			0.209		1.306		1.31	0.33		
443	183			0.401		2.033		2.12	0.52		
406	125			1 222		2.318		3.18	0.79		
364	83			0.479		2.332		2.45	0.61		
320	54			0.423		2.118		2.21	0.82		
240	17			0.262		1.506		1.53	0.76		
176	−3				3.436		9.731			9.27	4.63
129	−19				2.878		7.793			7.55	3.77
94	−36				2.554		6.668			6.55	3.27
71	−58				2.536		6.606			6.49	3.25
60	−89				2.896		7.856			7.61	1.90

Total Deformation In The Compression Zone = 3.80
Total Deformation In The Tension Zone = 16.82
Total Contribution To Surface Rut Depth = 3.80 + 16.82 = 20.62mm

Figure 19. Example of prediction of permanent deformation in a 25-cm bitumen-sand base.



Repeated Stresses (kN/m²)		T	N	Solutions Using Equations In Table 3				Results Using Equations In Table 3			
				ε_p at N = 1000		ε_p rate for N >1000		Total Vertical Permanent Deformation In The Element At N = 7000			
				I_c	I_t	S_c	S_t	ε_{pc}		ε_{pt}	
σ_1	σ_3			%	%	% /Log N	$\frac{\%}{N}$ (10^{-4})	%	mm	%	mm
477	398	25°C	7000	−0.267		−0.503		−0.69	−0.26		
460	274			0.123		0.978		0.95	0.23		
427	174			0.374		1.933		2.01	0.50		
382	94			0.505		2.432		2.56	0.96		
280	−26			5.722	5.722		17.668			16.32	8.16
196	−132				6.118		19.043			17.54	8.77
159	−273				7.990		25.543			23.31	5.82

Total Deformation In The Compression Zone = 1 43
Total Deformation In The Tension Zone = 22 75
Total Contribution To Surface Rut Depth = 1.43 + 22.75 = 24.18 mm

explained the major part of treatment-to-treatment variation in the experimental results.

4. Application of the predictive relationships derived in the study produced plausible solutions, but verification against observations of real performance remains to be done and is considered essential before reliable use in real design problems can be made.

5. The solutions provided by the predictive models indicated that (a) most of the permanent deformation occurs in the tension zone and (b) increasing the thickness of the treated sand layer has the effect of reducing its contribution to the total rut depth in the pavement system.

ACKNOWLEDGMENT

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