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.

REFERENCE

8. M.Y. Shahin and B.F. McCullough. Prediction of Low-Temperature and Thermal-Fatigue Cracking in Flexible Pavements. Center for Highway Research, Univ. of Texas at Austin, Res. Rept. No. 123-14, March 1970.

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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.

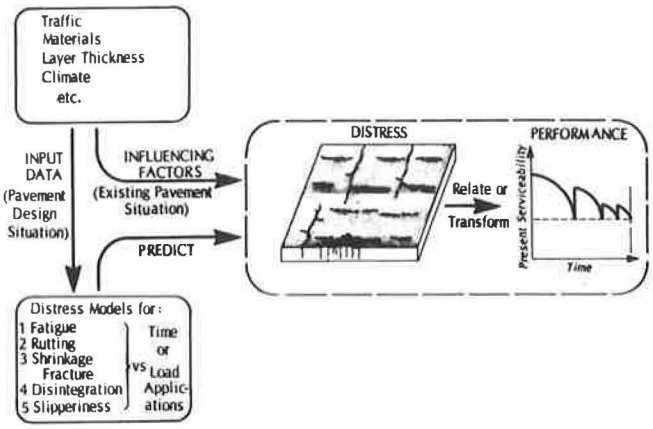
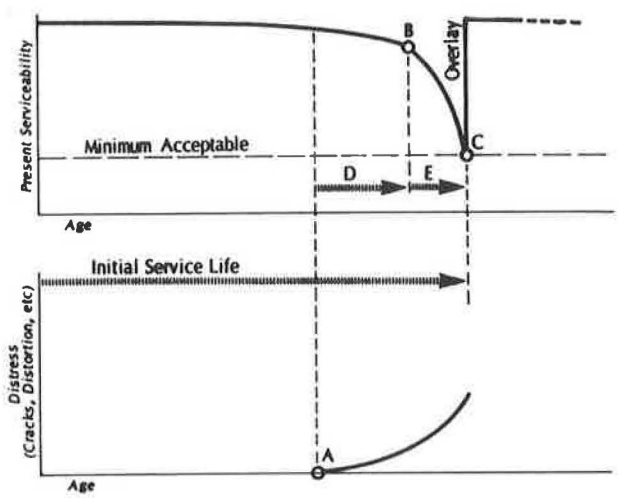


Figure 2. Time-delay nature of distress-performance relationships.



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.

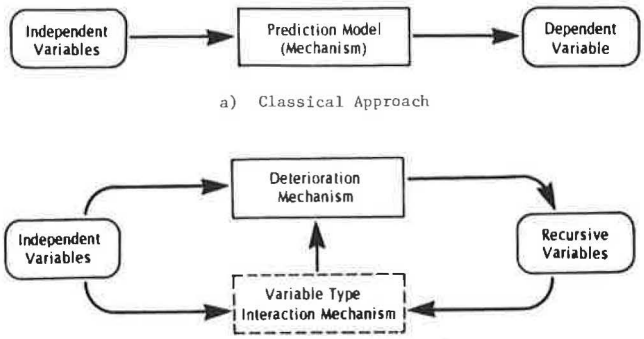
The use of roughness alone to estimate serviceability seems more logical since the user's perception of pavement serviceability is dominated by riding comfort (5). Therefore, by attempting to relate roughness to different combinations of the various amounts of distress measured during various periods 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.

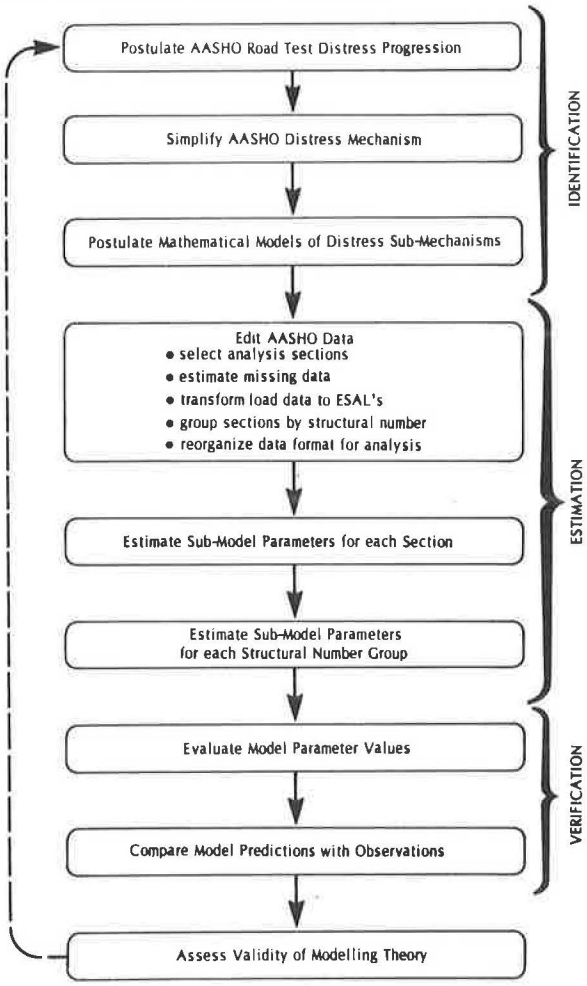
For example, once a certain level of cracking has been predicted, the effect of the same loads will increase pavement deterioration as the load-bearing capacity of the pavement structure is decreased. In another example, when roughness increases, the effect of the same traffic loads on the pavement structure 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.

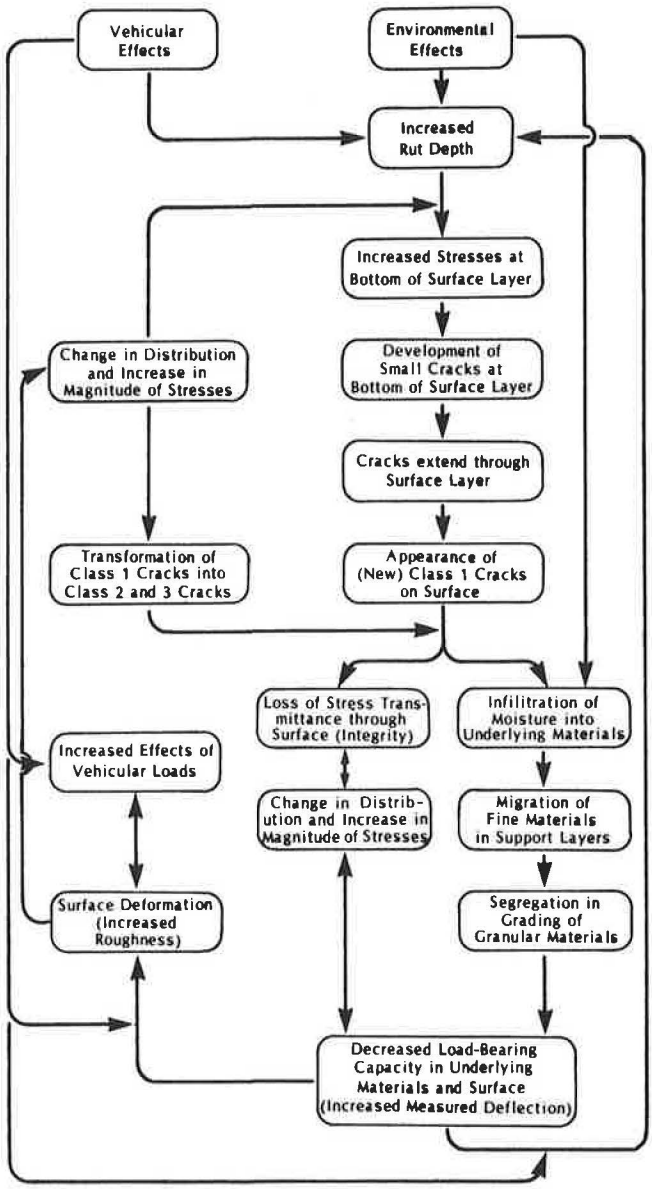
For this investigation, only flexible pavements were considered, although the main factorial design included both flexible and rigid structures. In the main factorial design, six levels of surface thickness, four levels of granular base thickness, five levels of subbase thickness, and 11 load configurations (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.

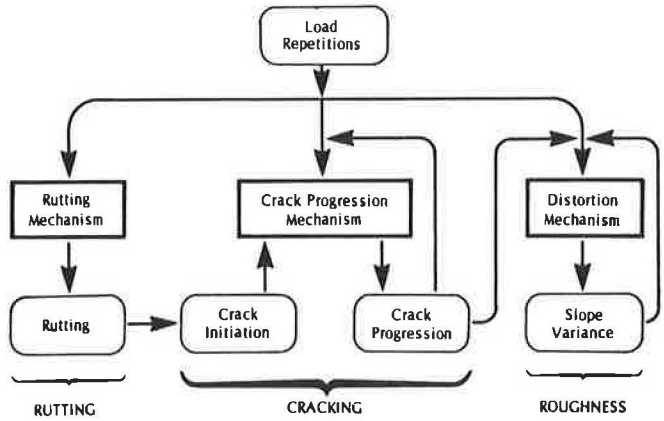


observations available in the individual time series restricts the number of variables that can be modeled and still maintain statistical significance. Since one type of subgrade and set of environmental conditions prevailed for all sections, the exclusion of environmental and structural capacity variables 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

Figure 6. Simplified AASHO distress mechanism.



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:

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

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

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

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 ($L_t < LI$; $LI \leq L_t$) and cracking level ($C_t = 0$; $0 < C_t < 5\%$; $5\% \leq C_t$).

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									
								Roughness			
		Rutting		Cracking				$L < LI_i$		$LI_i \leq L$	
SN Group	Cracking Interval	a	b	LI_i	c	d	e	f	g	f	g
1	C = 0							25.869	−0.609	53.744	−0.835
	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
2	C = 0							23.388	−0.558	40.396	−0.818
	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
3	C = 0							16.010	−0.371	70.130	−0.997
	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
4	C = 0							16.814	−0.421	47.892	−0.411
	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
5	C = 0							13.582	−0.417	24.719	−0.536
	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