Evaluation of Flexible Pavement Performance from Pavement Structural Response

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A model for predicting flexible pavement performance was developed from AASHO Road Test data. The pavement condition indicator selected for quantifying performance is pavement surface roughness, which correlated highly with pavement serviceability as perceived by the road user. In the development of the model, pavement performance was initially defined as the history of a pavement condition indicator(s) over time or with increasing axle load applications. Consequently, the performance model developed predicts the trend in pavement surface roughness with cumulative axle load applications. Since strain basins provide more information on the pavement response under load than just the maximum pavement response, indices determined from an evaluation of subgrade compressive and asphalt tensile strain basins were evaluated for their utility as performance predictors. The results obtained strongly indicate that strain basin indices are important predictors of the performance of AASHO flexible pavement sections.

A wide range of analytical procedures are used to determine the structural response of flexible pavements, including linear-elastic, viscoelastic, and finite-element methodologies. The specific performance-related pavement response variables (e.g., asphalt tensile strain, subgrade compressive strain) determined by these methodologies are used to design new pavement construction or pavement rehabilitation.

The task of designing pavements for transporting people and commodities is logically related to what pavement performance is perceived to be. One commonly accepted definition states that performance is the amount of service rendered by a pavement before reaching a failure condition (*1*). This definition is attractive in its simplicity, but has certain shortcomings for the same reason. Figure 1 illustrates what these shortcomings are.

Consider two pavement structures, A and B, and assume that the trend in each pavement's present serviceability index (PSI) with increasing load applications is as shown in Figure 1. From the definition given previously, the two pavements would be characterized as having the same performance, using terminal serviceability level PSI_1, since both pavements reached this terminal level of PSI after the same number of load applications. However, it is apparent that these two pavements provided different levels of service. Significantly different user and maintenance costs may be associated with pavements A and B. In addition, if PSI_2 is used as the terminal level of serviceability, pavement B would be considered to have the better performance, whereas if PSI_3 is used, then pavement A would be considered to have the better performance.

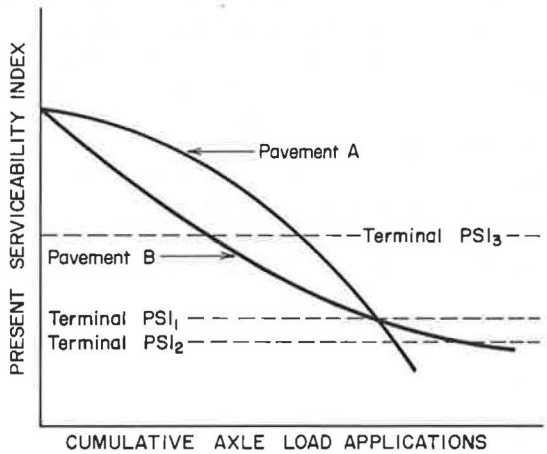

FIGURE 1 Conceptual serviceability histories for pavements A and B.

In view of the above shortcomings, a different definition of pavement performance was used in this research. Specifically, it was defined as the history of a pavement condition indicator(s) over time or with increasing axle load applications. This is believed to be a better definition than the one given previously for the following reasons:

1. A performance history is needed to consider the difference between two pavements that fail at the same time.
2. A performance history allows the comparison of two pavements that have not failed, rather than considering them as equally satisfactory.
3. A performance history is required for determining user and maintenance costs in a life-cycle cost analysis.

This definition was subsequently used to develop a performance model through an evaluation of the relationships between pavement performance and theoretical pavement response.

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RESEARCH SCOPE

The objective of this research was the evaluation of the relationships between pavement performance and structural response. Pavement failure was assumed to be a function of the response to vehicle loadings, and it was hypothesized that the variation in pavement performance can be explained from the corresponding variation in the theoretical structural response.

Although maximum asphalt tensile strain and maximum subgrade compressive strain are the most frequently used variables for predicting pavement performance, strain basin indices, developed from an evaluation of theoretical strain basins, were also examined for their usefulness as performance prediction variables. These quantities are analogous to deflection basin indices such as surface curvature index (SCI), base curvature index (BCI), or base damage index (BDI) (defined in Figure 2), which are used as indicators of pavement structural integrity. Strain basin indices are therefore related to theoretical strains at different locations within a pavement structure. Figure 3 shows a subgrade compressive strain basin for an 18,000-lb single-axle load.

The importance of strain basins in the evaluation of pavement performance is illustrated conceptually in Figure 4, which shows plots of the longitudinal distribution of subgrade compressive strains for two different pavements. If only the maximum subgrade compressive strain is considered, then the two pavements would be characterized as having the same pavement response under load. However, it is apparent from an examination of the strain basins in Figure 4 that this is not the case. The load distribution across the subgrade for pavement A is different from the load distribution for pavement B. Inasmuch as pavement performance is logically related to how the pavement responds under load, indices developed from an evaluation of strain basins may provide a better explanation of the variation in performance for different pavement structures.

Performance data from flexible pavement sections at the AASHO Road Test (*2*) were used in the development of the performance model presented herein. Pavement surface roughness, as measured by slope variance (*SV*), was the pavement condition indicator selected for modeling pavement performance. This pavement condition indicator is strongly correlated with riding quality as perceived by the road user. This is evident in Figure 5, which shows a plot of present service-

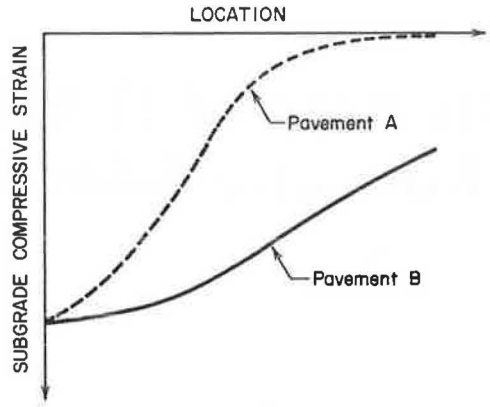

FIGURE 4 Conceptual subgrade compressive strain basins for pavements A and B.

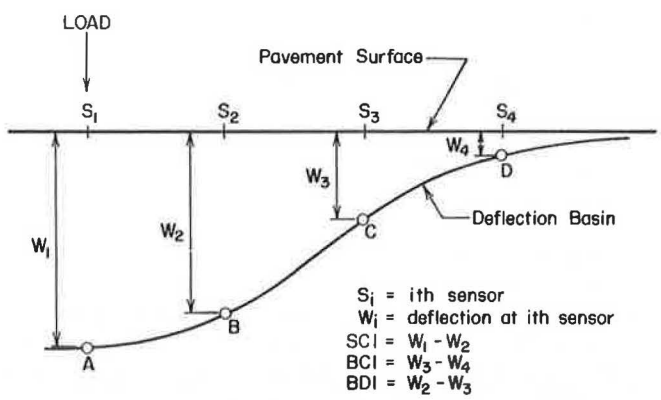

FIGURE 2 Example of surface deflection basin.

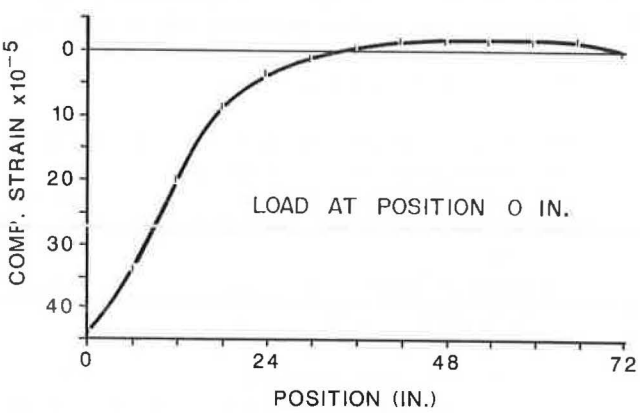

FIGURE 3 Subgrade compressive strain basin for an 18-kip single-axle load.

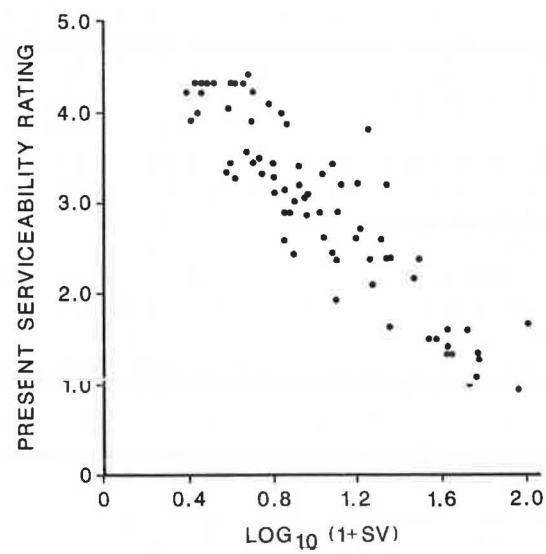

FIGURE 5 Present serviceability rating vs. $\log_{10}(1 + SV)$.

ability ratings vs. $\log_{10}(1 + SV)$, an indicator of pavement roughness. The present serviceability ratings were made by a panel of highway users who rated, on a scale of 0 to 5, the riding quality of 74 selected flexible pavement sections at the time of the AASHO Road Test (*2*).

For this research therefore, roughness was used in modeling pavement performance. It was beyond the scope of the research to develop performance models using other pavement condition indicators such as cracking and rutting. In addition, the evaluation of pavement structural response was made using linear elastic-layer theory, which is widely used for the analysis of pavement structures. The stress dependency of the moduli of unbound pavement materials was considered in the computation of pavement strains. Subgrade compressive and asphalt tensile strain basins were evaluated in the study, and statistical regression techniques were used to evaluate relationships between pavement performance and pavement response.

LOGISTIC REGRESSION ANALYSIS OF AASHO ROAD TEST DATA

Performance data from the AASHO Road Test (*2*) were analyzed to develop an equation for identifying pavement designs that would likely lead to premature pavement failure. Examination of AASHO Road Test data revealed that certain flexible pavement sections experienced early failure. These sections did not last beyond the first spring season, after traffic operations began in October 1958, and sustained less than 100,000 18-kip ESALs prior to reaching a terminal serviceability level of 1.5.

Figure 6 shows a comparison of the performance history of a section that failed prematurely with the performance history of a section that lasted the duration of the AASHO Road Test. For most of the sections that failed early, the PSI values were adequate until a sharp decrease in serviceability was observed at the onset of the spring season. This sudden loss in serviceability was most likely due to the adverse subsurface conditions that existed during this period. Because of the thawing that occurred at this time of the year, subgrade support conditions were poor, leading to the development of pavement distress, primarily cracking and rutting.

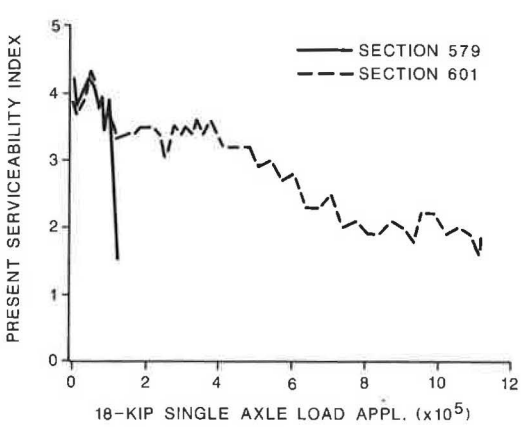

FIGURE 6 Comparison of PSI histories of sections 579 and 601.

In this particular study, AASHO Road Test pavements that experienced premature failure (i.e., sustained less than 100,000 18-kip ESALs) were categorized as Class 1 sections, and those that lasted longer were categorized as Class 2. Since this research was concerned with evaluating the performance of flexible pavements designed for high-volume traffic conditions, the procedure developed for discriminating between Class 1 and Class 2 pavements was used to sort the AASHO flexible pavement sections into the aforementioned categories. Only the performance of Class 2 sections was subsequently evaluated.

A statistical technique, known as logistic regression, was used to develop an equation for discriminating between Class 1 and Class 2 pavement sections. The logistic model for the case where the dependent variable is binary (0 or 1) is given by the following equation:

$$\Pr[Y = 1] = 1/[1 + \exp(-\mathbf{X}'\boldsymbol{\beta})] \quad (1)$$

where

$\Pr[Y = 1]$ = probability of premature failure,
$\mathbf{X}'$ = transpose of the vector of independent variables, and
$\boldsymbol{\beta}$ = vector of model parameters.

In the development of the logistic regression equation from AASHO Road Test data, the binary dependent variable Y_i was set to 1 for flexible pavement sections that failed early. Sections that failed early were defined as those that did not last beyond the first spring season and sustained less than 100,000 18-kip ESALs. For all other sections, Y_i was set equal to 0. The independent variables included in the analysis were the surface, base, and subbase layer thicknesses, and the theoretical values of subgrade compressive strain, calculated using multilayer linear elastic theory. Subgrade compressive strain was calculated using the applied axle load for a given test section. The subgrade strain directly beneath one tire was calculated.

Table 1 summarizes the material properties assumed for the various layers of the AASHO flexible pavement sections. The material properties are representative of average yearly conditions at the AASHO test site. The modulus of the asphalt concrete mix was obtained at a test temperature of 70°F (*3*).

A computer program called NEL1 was used to calculate the compressive strain at the top of the subgrade for each of the 284 AASHO flexible pavement sections included in the experiment. This program was developed by Luhr and McCullough (*4*) and is a modification of the BISAR program (*5*). NEL1 considers the stress dependency of the resilient modulus of unbound pavement materials in the calculation of stresses, strains, and displacements within the pavement structure. This is accomplished through an iterative application of linear elastic-layer theory to get stress-compatible moduli.

The theoretical values of subgrade compressive strain and the layer thicknesses of the flexible pavement sections were used as independent variables in a logistic regression analysis. A stepwise nonlinear regression procedure (*6*) was used to determine the coefficients of the logistic model given by Equation 1. Table 2 summarizes the results of the regression analysis, showing the independent variables that entered at each

TABLE 1 MATERIAL PROPERTIES OF AASHO FLEXIBLE PAVEMENT SECTIONS

Material	Modulus (psi)
1. Asphalt concrete	450,000
2. Crushed limestone base	$4000\,\theta^{0.6}$
3. Sand-gravel subbase	$5400\,\theta^{0.6}$
4. Subgrade	$27{,}000\,\sigma_d^{-1.06}$

step of the procedure, and the fraction of concordant pairs. The final logistic regression equation is:

$$\Pr[Y = 1] = 1/\Big(1 + \exp\{-[16.586 - 0.300\,H_1 - 0.722\,H_2 - 0.353\,H_3 + 3.824\,\log_{10}(\varepsilon_{sg})]\}\Big) \quad (2)$$

where

$\Pr[Y = 1]$ = probability of premature failure,
H_1 = asphalt concrete thickness (in.),
H_2 = base thickness (in.),
H_3 = subbase thickness (in.), and
ε_{sg}. = subgrade compressive strain.

The fraction of concordant pairs is a statistic that measures the predictive ability of the model. This statistic can have a maximum value of 1.0 indicating a model with excellent ability to classify observations into the different levels of the dependent variable. For the logistic regression equation developed, the fraction of concordant pairs is equal to 0.93, indicating that a model with good predictive ability has been obtained.

The results of the analysis summarized in Table 2 reveal that, for the independent variables considered, the base and subbase layer thicknesses initially provided the largest contribution to the predictive ability of the model. As can be seen in the table, the fraction of concordant pairs increased from 0.500 to 0.805 with the addition of base thickness as an independent variable, and attained a value close to 0.90 with just the base and subbase thicknesses in the logistic regression equation. The significant ability of these two variables to discriminate between Class 1 and Class 2 pavements reflects the fact that of the 84 flexible pavement sections that failed early during the Road Test, 56 had either no base and/or subbase. This is indicated in Figures 7 and 8, which show the distributions of base and subbase layer thicknesses drawn according to the level of the pavement class variable.

Most of the Class 1 pavements failed during the initial spring season at the AASHO Road Test. Considering the adverse subgrade support conditions that existed during this time period, the absence of base and/or subbase layers would further accelerate pavement distress and lead to premature failure.

The results of the analysis shown in Table 2 also indicate that the contribution of asphalt concrete thickness to the fraction of concordant pairs is small relative to those for the base and subbase layer thicknesses. However, this observation does not imply that asphalt concrete thickness has little effect on

TABLE 2 RESULTS OF STEPWISE LOGISTIC REGRESSION ANALYSIS

Step	Variable entered	Variable removed	Fraction of concordant pairs
0	intercept	-	0.500
1	base thickness	-	0.805
2	subbase thickness	-	0.896
3	$\log_{10}(\epsilon_{sg})$	-	0.926
4	surface thickness	-	0.930

ϵ_{sg} = compressive strain at the top of the subgrade

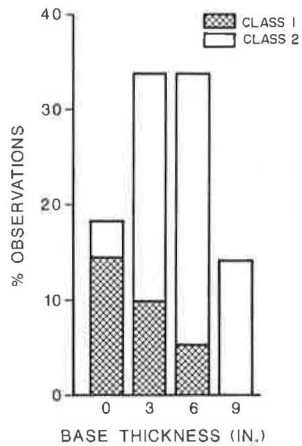

FIGURE 7 Distribution of base thicknesses of AASHO flexible pavement sections.

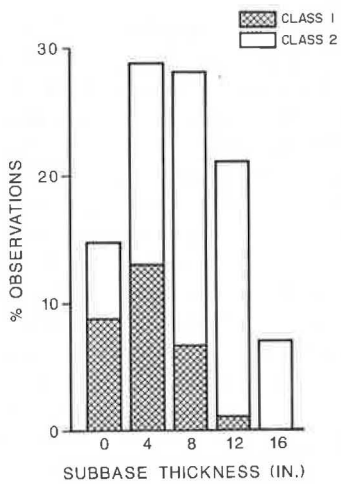

FIGURE 8 Distribution of subbase thicknesses of AASHO flexible pavement sections.

pavement performance. Rather, the results presented should be viewed relative to the purpose for conducting the analysis, which was to develop an equation for discriminating between two different classes of pavement sections. For this purpose, the base and subbase thicknesses are variables that better discriminate between Class 1 and Class 2 sections. A plausible explanation for this was given previously, i.e., that most of the premature failures were associated with sections that were built without base and/or subbase layers. It is expected that in the evaluation of the performance of sections categorized as Class 2, the thickness of the asphalt concrete layer would have a significant effect on pavement performance.

DEVELOPMENT OF PERFORMANCE MODEL

The logistic regression equation presented above was used to select the AASHO flexible pavement sections to include in the performance modeling phase of the research effort. As part of the development of the performance model, an evaluation was made to identify trends in pavement performance with increasing axle load applications. In the evaluation, the variation in roughness with increasing axle load applications was examined for each of the AASHO flexible pavement sections categorized as Class 2 from the logistic regression analysis. From this examination, the performance trends illustrated in Figure 9 were identified.

Efforts were made to identify a model that would define the five trends shown in Figure 9. Three different models were evaluated: The Weibull (Eq. 3); the logistic (Eq. 4); and the cubic, i.e., third-degree polynomials (Eq. 5).

$$y = \beta_3\left(1 - \exp\{-[(N - \beta_0)/\beta_1]^{\beta_2}\}\right) \quad (3)$$

$$y = \beta_0/[1 + \beta_1 \exp(-\beta_2 N)] \quad (4)$$

$$y = \beta_0 + \beta_1 N + \beta_2 N^2 + \beta_3 N^3 \quad (5)$$

where

$y = \log_{10}(1 + SV)$,
N = cumulative number of axle load applications, and
β_i = model parameters determined from regression analysis.

Using linear as well as nonlinear regression techniques, efforts were made to determine whether any of the models given in the above equations fit all of the five trends identified in Figure 9. It was found that the cubic model could fit the five performance trends shown. Consequently, in the development of the performance model, the form of the model was taken to be a cubic or third-degree polynomial.

The next step in the development of the performance model was the evaluation of the parameters of the cubic model. Pavement performance can be predicted if, for any given design, the parameters of this model can be specified. Consequently, it was essential to develop prediction equations for the parameters of the cubic model. In connection with this, it was reasonable to assume that the model parameters are functions of certain attributes of the pavement design. Consistent with the hypothesis for this research, an evaluation of the relationships between the model parameters and pavement structural response variables (strains and strain basin indices) was conducted.

For each of the AASHO flexible pavement sections included in the analysis, the cubic model was fitted to the observed performance data to estimate the model parameters for each test section. Inasmuch as there were 202 sections to analyze, a program was written to accomplish this task on a personal computer.

An evaluation of the relationships between the fitted model parameters and pavement structural response variables was then conducted. Each model parameter (i.e., β_1, β_2, β_3) was plotted vs. pavement structural response variables such as compressive strain at the top of the subgrade, tensile strain at the bottom of the asphalt layer, and strain basin indices. Compressive strain basins at the top of the subgrade, and tensile strain basins at the bottom of the asphalt layer were determined using computer program NEL1 with the material properties shown in Table 3.

In the development of the performance model, the evaluation of seasonal effects was considered important. However, after a careful examination of AASHO Road Test data, it

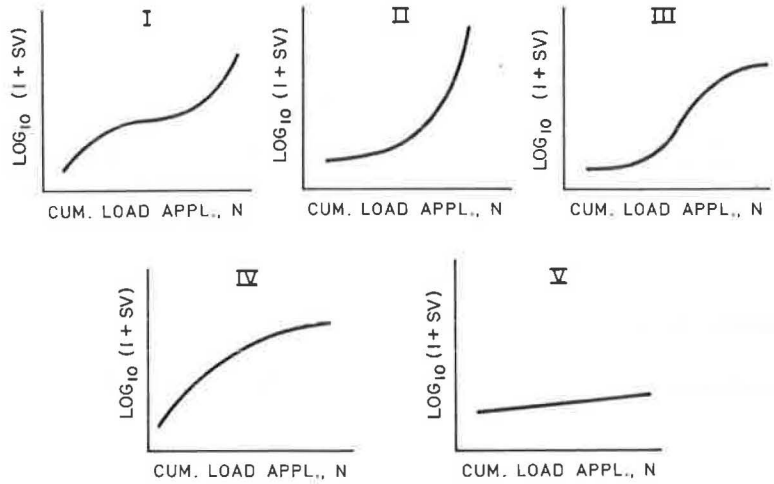

FIGURE 9 Observed performance trends of AASHO flexible pavement sections.

was found that an analysis of seasonal effects cannot be made since the rate of loading was not systematically varied during the Road Test. Consequently, it was not possible to separate the effects of load and season on the observed trends in pavement performance. The decision was therefore made to use material properties representative of average yearly conditions at the AASHO Road Test site, and to model the entire performance history exhibited by each Class 2 section during the duration of the Road Test.

From the strain basins determined, the indices shown in Table 3 were evaluated. As noted previously, strain basin indices are analogous to deflection basin indices, which are currently used by many highway agencies as indicators of pavement structural integrity. These indices are related to strains at different locations within the pavement structure. Inasmuch as the strain basin provides more information on the response of the pavement under load than just the maximum pavement response, strain basin indices were evaluated for their utility as performance prediction variables.

Figures 10 and 11 show conceptual illustrations of compressive strain basins at the top of the subgrade for single and tandem axles, respectively. These were the axle configurations that were considered at the AASHO Road Test. For each axle configuration, the elements of the strain basin used for calculating the subgrade strain basin indices presented in Table 3 are shown. Similar illustrations can be drawn for tensile strain basins at the bottom of the asphalt layer.

Through regression analysis, prediction equations for β_1, β_2, and β_3 of the cubic model were developed. Table 4 shows the prediction equations for the cubic model parameters. It was found that the subgrade strain basin index V_2 provided equations with the highest coefficients of determination.

No prediction equation was developed for β_0. Physically, this parameter represents the initial surface roughness after construction, and is an input to the pavement design process that is provided by the highway engineer. Alternatively, β_0 may be one of the specifications established for the construction of flexible pavement structures.

The prediction equations shown in Table 4 were subsequently evaluated by comparing predicted performance trends with the observed performance trends at the AASHO Road Test. As may be discerned from Figure 12, the results obtained were not very encouraging. It was found that the predicted performance trend usually diverged from the observed performance trend. The divergence is very significant. Observed values of pavement roughness [$\log_{10}(1 + SV)$] for bituminous sections ranged from 0.18 to 2.4 at the AASHO Road Test. However, because of the significant disparity between the predicted and observed values for pavement roughness, the observed performance trend appears to be just a horizontal line passing through the zero point of the y axis.

The predictions for pavement roughness were so unrealistic that the possibility of arithmetic or programming errors in the analysis was initially investigated. However, a careful review showed that no such errors were committed. The predicted parameters of the cubic model were simply giving unacceptable results. Consequently, efforts were made to improve the accuracy of the prediction equations for the cubic model parameters (Table 4), and new equations were subsequently developed. However, the results obtained were again very discouraging; many predicted performance trends still diverged unreasonably from the observed trends.

The difficulties encountered may be attributable to the extreme sensitivity of the predictions from the cubic model to deviations in the model parameters, primarily those that are multipliers of the X^2 and X^3 terms (i.e., β_2 and β_3). Even though the cubic model adequately fitted the observed performance histories, the model coefficients, particularly those that are multipliers of the x^2 and x^3 terms, have to be predicted very accurately in order to come up with performance predictions comparable with the observed data. Deviations between the predicted and fitted coefficients were significantly magnified by the higher order terms leading to unrealistic predictions of pavement performance.

In view of the difficulties encountered with the cubic model, it was decided that further modeling involving the cubic polynomial would not be productive, and that working with another model would be the best direction to follow. Consequently, efforts were made to identify another mathematical equation for modeling the observed AASHO performance trends. Previously, plots showing the variation in pavement surface roughness, as quantified by $\log_{10}(1 + SV)$, with cumulative

TABLE 3 EVALUATION OF STRAIN BASIN INDICES

Strain Basin Index		Definition
1.	V_1	$\varepsilon_{sg1} - \varepsilon_{sg_{max}}$
2.	V_2	$\varepsilon_{sg2} - \varepsilon_{sg_{max}}$
3.	V_3	$\varepsilon_{sg3} - \varepsilon_{sg_{max}}$
4.	V_4	$\varepsilon_{sg4} - \varepsilon_{sg_{max}}$
5.	V_5	$\varepsilon_{sg2} - \varepsilon_{sg1}$
6.	V_6	$\varepsilon_{sg3} - \varepsilon_{sg2}$
7.	V_7	$\varepsilon_{sg4} - \varepsilon_{sg3}$
8.	V_8	$\varepsilon_{sg3} - \varepsilon_{sg1}$
9.	V_9	$\varepsilon_{sg4} - \varepsilon_{sg2}$
10.	V_{10}	$\varepsilon_{sg4} - \varepsilon_{sg1}$
11.	T_1	$\varepsilon_{ac_{max}} - \varepsilon_{ac1}$
12.	T_2	$\varepsilon_{ac_{max}} - \varepsilon_{ac2}$
13.	T_3	$\varepsilon_{ac_{max}} - \varepsilon_{ac3}$
14.	T_4	$\varepsilon_{ac_{max}} - \varepsilon_{ac4}$
15.	T_5	$\varepsilon_{ac1} - \varepsilon_{ac2}$
16.	T_6	$\varepsilon_{ac2} - \varepsilon_{ac3}$
17.	T_7	$\varepsilon_{ac3} - \varepsilon_{ac4}$
18.	T_8	$\varepsilon_{ac1} - \varepsilon_{ac3}$
19.	T_9	$\varepsilon_{ac2} - \varepsilon_{ac4}$
20.	T_{10}	$\varepsilon_{ac1} - \varepsilon_{ac4}$

$(\varepsilon_{sg})_{max}$ = maximum subgrade compressive strain directly underneath the tire load

$(\varepsilon_{sg})_i$ = subgrade compressive strain located along the longitudinal direction and at a distance of 'i' feet from the maximum

$(\varepsilon_{ac})_{max}$ = maximum asphalt tensile strain directly underneath the tire load

$(\varepsilon_{ac})_i$ = asphalt tensile strain at a distance of 'i' feet from the maximum

number of load applications, were examined to identify a mathematical equation for modeling the performance of AASHO flexible pavement sections. In the reevaluation of observed performance trends, new plots showing the variation in pavement roughness with the logarithm (base 10) of the cumulative number of load applications were examined. It was realized that the trends observed would be affected by transformations to the variables being plotted. Consequently, a logarithmic transformation of the cumulative number of load applications was applied to see if a different set of performance trends would be obtained. It was found that with this transformation, the new trends observed could be modeled by the hyperbolic equation

$$y = (\beta_0 + \beta_1 x)/(1 + \beta_2 x) \tag{6}$$

where

y = pavement surface roughness, $\log_{10}(1 + SV)$,
x = logarithm (base 10) of cumulative number of load applications, and
β_i = parameters of the hyperbolic model.

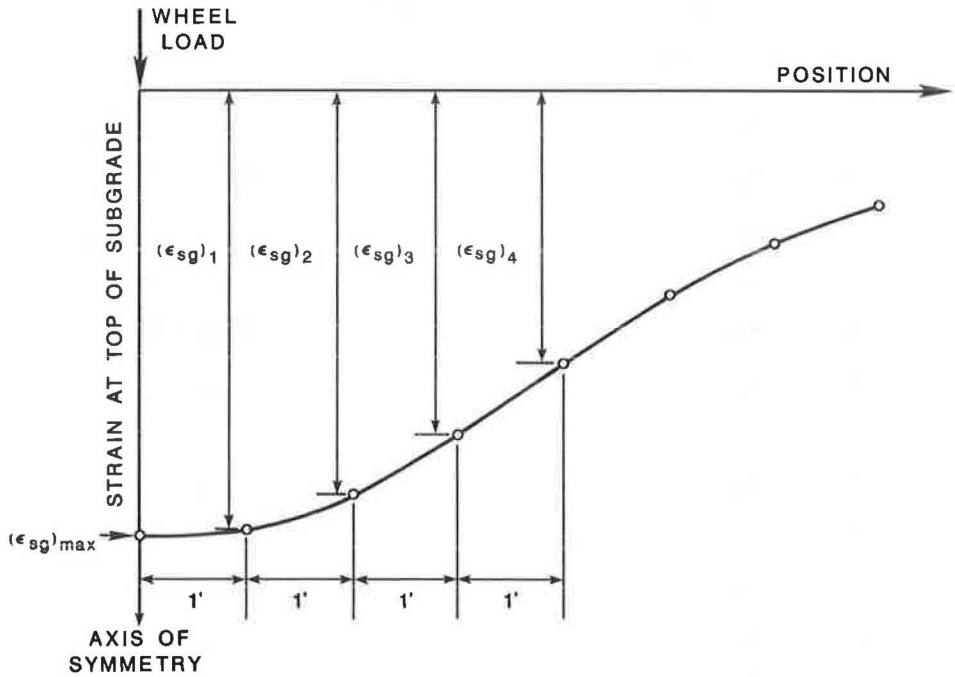

FIGURE 10 Conceptual illustration of subgrade compressive strain basin for single-axle configuration.

Using nonlinear regression, the parameters of the hyperbolic model were estimated by fitting the model to the observed performance data for each AASHO flexible pavement section included in the modeling phase of the research effort. Prediction equations for the fitted model parameters (β_1 and β_2) were subsequently developed using stepwise multiple linear regression. No prediction equation was established for β_0 for reasons mentioned previously. The prediction equations are as follows:

$$\beta_1 = -0.035 - 0.220\,\beta_0 - 0.035 \log_{10} V_3 - 0.050 \log_{10}(1 + H_1) \qquad R^2 = 0.56 \quad N = 202 \text{ obs.} \tag{7}$$

$$\beta_2 = -0.354 + 1.232\,\beta_1 + 0.269\sqrt{\beta_0} - 31.958\,V_5 - 0.026 \log_{10} T_2 + 0.007 \log_{10}(1 + H_2) \qquad R^2 = 0.93 \quad N = 202 \text{ obs.} \tag{8}$$

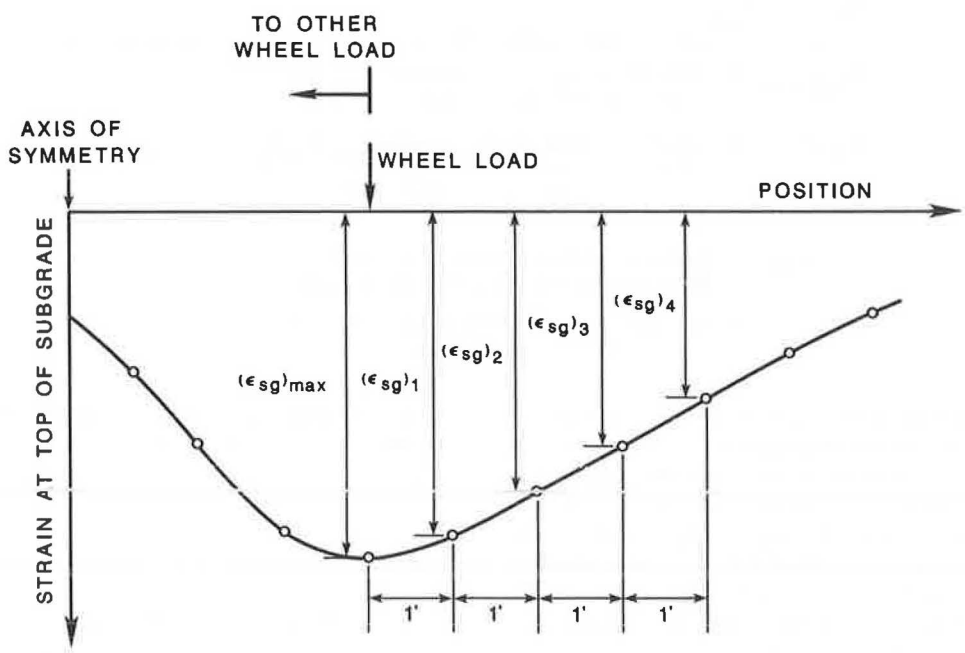

FIGURE 11 Conceptual illustration of subgrade compressive strain basin for tandem-axle configuration.

TABLE 4 PREDICTION EQUATIONS FOR CUBIC MODEL PARAMETERS

$$\beta_1 = -0.109 + 0.959\ V_2 \qquad \text{for } V_2 < 0.700$$

$$= -0.109 + 0.959\ V_2 + 9.143\ (V_2 - 0.700) - 4.143\ (V_2 - 0.700)^2 \qquad \text{for } V_2 \geq 0.700$$

$R^2 = 0.56$ N = 202 obs.

$$\beta_2 = 0.500 - 2.146\ V_2 \qquad \text{for } V_2 < 0.700$$

$$= 0.500 - 2.146 V_2 - 35.771(V_2 - 0.700) + 13.895\ (V_2 - 0.700)^2 \qquad \text{for } V_2 \geq 0.700$$

$R^2 = 0.59$ N = 202 obs.

$$\beta_3 = -0.616 + 2.253\ V_2 \qquad \text{for } V_2 < 0.700$$

$$= -0.616 + 2.253\ V_2 + 31.474\ (V_2 - 0.700) - 9.956\ (V_2 - 0.700)^2 \qquad \text{for } V_2 \geq 0.700$$

$R^2 = 0.62$ N = 202 obs.

$V_2 = \varepsilon_{sg2} - \varepsilon_{sgmax}$ (x 10^{-3})

ε_{sgmax} = maximum subgrade compressive strain directly underneath the tire load

ε_{sg2} = compressive strain at the top of the subgrade located along the longitudinal direction and at a distance of 2 feet from the maximum

where

β_i = the ith parameter of the hyperbolic model;
V_3, V_5, T_2 = strain basin indices defined in Table 3;
H_1 = thickness of asphalt concrete layer (in.); and
H_2 = thickness of base layer (in.).

To evaluate the accuracy of the performance predictions from these equations, observed vs. predicted trends were compared. It was found that the predicted performance trends were much more reasonable than those obtained with the prediction equations for the cubic model parameters. In addition, a root-mean-square (RMS) statistic, indicating the accuracy of the predictions, was calculated from the following equation:

$$\text{RMS} = \left[\sum_{i=1}^{n}(y - y')^2/n\right]^{1/2} \tag{9}$$

where

y = observed $\log_{10}(1 + SV)$,
y' = predicted $\log_{10}(1 + SV)$, and
n = total number of observations.

The RMS statistic for the performance predictions was 0.24 on the basis of 5,895 observations. A similar statistic calculated from the observed performance data for the replicate sections at the AASHO Road Test was 0.19 for 767 observations. (Replicate sections were identical pavement sections constructed at the AASHO Road Test.) Thus the RMS statistic for the performance model compares favorably with the RMS statistic for the replicates, which gives a measure of the pure error in observed pavement performance.

Figure 13 illustrates how the predictions from the model compare with the observed values for pavement roughness. The predictions generally compare favorably with the observed roughness data as reflected by the dark region around the line of equality. The correlation coefficient between the predicted and observed $\log_{10}(1 + SV)$ was determined to be 0.59. In contrast, the correlation coefficient for the observed $\log_{10}(1 + SV)$ between replicates was found to equal 0.44. The fact that a higher correlation coefficient was obtained from the model predictions reflects the smoothing effect of the curve-fitting conducted as part of the model development. In addition, it further indicates that a performance model with reasonable predictive ability has been developed.

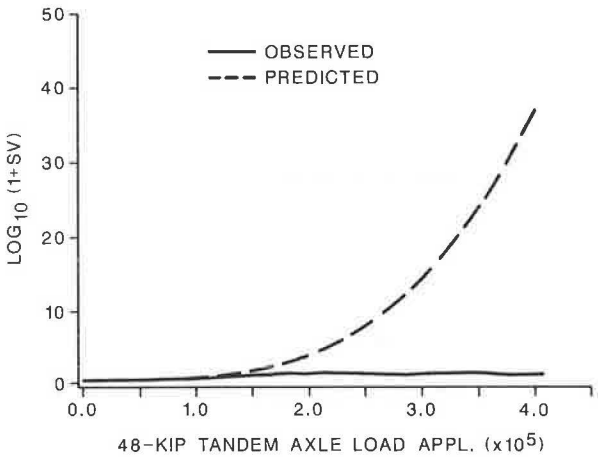

FIGURE 12 Comparison of observed and predicted performance trends for section 262 using the cubic model.

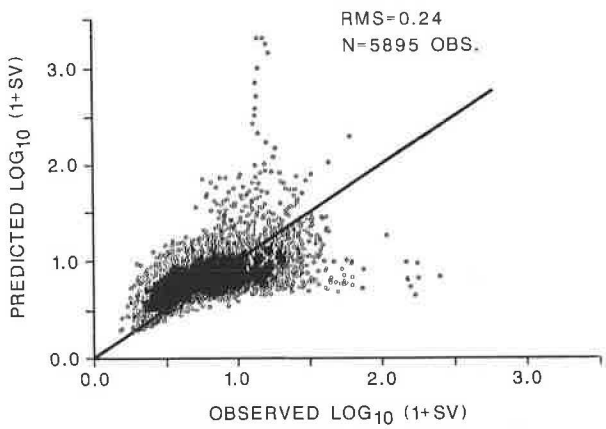

FIGURE 13 Comparison of predicted $\log_{10}(1 + SV)$ from the hyperbolic model with the observed $\log_{10}(1 + SV)$.

SUMMARY

A performance model was developed in this paper for predicting the trend in pavement surface roughness with cumulative number of load applications. In view of the difficulties encountered with the cubic model, a reevaluation of observed AASHO performance trends was conducted in order to identify another mathematical function for modeling the performance of AASHO flexible pavement sections. It was found that the predictions from the cubic model were extremely sensitive to deviations in the model coefficients, primarily those that are multipliers of the X^2 and X^3 terms. Consequently, the model coefficients have to be predicted very accurately in order to come up with performance predictions comparable with the observed performance. The reevaluation of observed AASHO performance histories resulted in the selection of the hyperbolic equation for modeling the performance of AASHO flexible pavement sections.

Prediction equations for the parameters of the hyperbolic model were subsequently developed. It was found that the hyperbolic equation adequately modeled the observed performance of AASHO flexible pavement sections. However, it should be realized that the performance model was developed using pavement surface roughness as the condition indicator, and that other mathematical functions may be more appropriate for other condition indicators.

Consistent with the hypothesis for this research, an evaluation of the relationships between the hyperbolic model parameters and pavement structural response variables (strains and strain basin indices) was conducted. Through stepwise multiple linear regression, prediction equations were developed that included subgrade and asphalt strain basin indices as independent variables. In the stepwise regression analysis, both the maximum asphalt tensile strain and maximum subgrade compressive strain were allowed to enter into the prediction equations in addition to strain basin indices. The fact that strain basin indices were selected as independent variables reflects the importance of the strain basin for evaluating pavement performance, and provides a strong indication of the utility of strain basin indices as performance predictors.

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