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Structural Design of Flexible Pavements: A Simple Predictive System

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During the past two decades, much effort has gone into the development of rational pavement design procedures that are intended to be integrated into a more general framework. The Federal Highway Administration has developed a computer program package known as VESYS II to predict the structural responses and hence the integrity of flexible pavements. The program is quite formidable for the local design engineer. Therefore, a simple predictive system is needed to be widely used for the structural design of flexible pavements. A simple computer program package is presented that includes (a) regression formulas for tensile strain and rut depth computations, cracking prediction, and evaluation of the rut depth variance; (b) modification and calibration of the American Association of State Highway Officials (AASHO) Road Test serviceability model, where rut depth variance replaces the slope variance; (c) seasonal (monthly) characterization of pavement materials and discrete representation of axle-load distribution; and (d) special treatment for overlay analysis. The procedure is illustrated and the results are discussed. The good agreement between the results and measured values and the simplicity of the program make it very attractive. It could be programmed on a desk (micro) computer.

During the past two decades, much effort has gone into the development of rational pavement design procedures that were intended to be integrated into a more general framework. The Federal Highway Administration (FHWA) (1) has developed a computer program package known as VESYS II (and its modifications and extensions VESYS IIM, VESYS A, and VESYS G) to predict the structural responses and hence the integrity of flexible pavements. The program is based on an advanced viscoelastic analysis that is, however, a rather significant departure from more conventional systems. The implementation and use of the VESYS package seems appropriate for a statewide study, but it appears quite formidable for the design engineer. At the same time, pavement structural subsystems have been developed for the Transportation Research Board (2). The computer programs--one for fatigue cracking and permanent deformation (named PDMAP) and one for low-temperature cracking (named COLD)--were proposed for implementation and field calibration. The PDMAP package does not include the synthesis of the two subsystems and does not predict pavement performance.

This paper presents the integration of the fatigue and permanent deformation subsystems into a computer program package. The proposed system is simple, reliable, and also general, which facilitates its use by most people who deal with pavement design and maintenance. It includes the following:

1. Modification and calibration of the American Association of State Highway Officials (AASHO) Road Test serviceability model to enable pavement performance prediction by using computed damages,
2. Cracking prediction based on the commonly used fatigue law and probabilistic considerations to express mechanistic variables into cracked areas,
3. Permanent deformation prediction based on quasi-elastic analysis,
4. Regression formulas and closed-form probabilistic solutions for evaluation of the variables involved in the performance model (the probabilistic solutions are similar to those used in VESYS G),
5. Overlay application analysis to permit pavement maintenance strategy studies, and
6. Seasonal (monthly) characterization of pavement materials and discrete representation of axle-load distribution.

SERVICEABILITY MODEL

The AASHO Road Test serviceability model relates serviceability index to variables that describe pavement damage, i.e., cracking, patching, rutting, and slope variance (3). The AASHO model reads as follows:

$$\mathrm{PSI} = 5.03 - 1.91 \log_{10} (1 + \overline{SV}) - 1.38\,\overline{RD}^2 - 0.01 \sqrt{\overline{C} + \overline{P}} \quad (1)$$

$R^2 = 0.84$

where

PSI = present serviceability index,
$\overline{\text{SV}}$ = average slope variance (10^6 radian),
$\overline{\text{RD}}$ = average rut depth (in),
$\overline{C}$ = cracking ($ft^2/1000\ ft^2$),
$\overline{P}$ = patching ($ft^2/1000\ ft^2$), and
R = correlation coefficient.

The model was used in its original form in the VESYS IIM program. Its implementation required the prediction of all the basic variables involved: $\overline{\text{SV}}$, $\overline{\text{RD}}$, and $\overline{C}$. [The variable $\overline{P}$ (patching) was dropped from Equation 1, as it was taken into account through the variable $\overline{C}$ (cracking).] It should be noted that only two mechanistic subsystems exist for the prediction of the three variables mentioned above. Cracking and rutting are readily dealt with by the fatigue and permanent deformation subsystems, respectively, while the slope variance variable, which is the result of the variability of the material in the longitudinal direction, required additional development. Its prediction in VESYS II is based on the stochastic variation of the rut depth (1) and on assumptions that concern the spatial autocorrelation function. Rauhut and others (4,5) used an approximation for the relation between slope variance and rut depth and its variance as follows:

$$\overline{\text{SV}} \simeq k\ \text{Var}\ [\text{RD}] \tag{2}$$

where k is the coefficient derived from linear regression with intercept forced through the origin, and Var [RD] is the rut depth variance.

The regression for all data from all four states was very poor. Substituting Equation 2 into Equation 1 corresponds, in fact, to a change of variables in Equation 1. Better results would be obtained by handling the original problem, i.e., deriving the serviceability model from raw data by using predictable variables only. The new serviceability model is then

$$\text{PSI} = a_0 + a_1 \log_{10} (1 + a_2\ \text{Var}\ [\text{RD}]) + a_3\ \overline{\text{RD}}^{a_4} + a_5\ (\overline{C} + \overline{P})^{a_6} \tag{3}$$

where a_0 to a_6 are the new regression coefficients. A nonlinear regression analysis was made on the four test sections of the AASHO model. The following equation was obtained by trial and error:

$$\text{PSI} = 4.436 - 1.686 \log_{10} (1 + 350\ \text{Var}\ [\text{RD}]) - 0.881\ \overline{\text{RD}}^{2.5} - 0.031\sqrt{\overline{C} + \overline{P}} \tag{4}$$

$R^2 = 0.80$

In this equation the effect of cracking became significant, while in Equation 1 the cracking term could be dropped without affecting the correlation coefficient. Furthermore, the intercept coefficient a_0 is very close to the average value of the pavement serviceability after construction (4.44 as compared with 4.20). Therefore, the new Equation 4 might be of greater engineering significance than Equation 1, even for a lower correlation coefficient ($R^2 = 0.80$ as compared with 0.84).

FATIGUE SUBSYSTEM

All fatigue subsystems presented in the literature are based on (a) computing tensile stresses or strains at the bottom of the surface layer, (b) evaluating the pavement service life from the relation between number of repetitions to failure (N_f), and (c) applied stress or strain. Few fatigue subsystems extended the analysis to predict the cracked area based on field correlation (2) or probabilistic solutions (1,6).

Computation of Tensile Strain at Bottom of Asphalt Concrete

The tensile strain at the bottom of the asphaltic concrete layer has been computed for a dual wheel load with a center-to-center distance between wheels of three times the radius of contact. The computation was made by using the BISAR program of the Shell Company and at two points: under one wheel of the dual load and between the wheels. Comparison of these results for radial and tangential strains under one wheel of the dual load and between the wheels shows that, in general, the tangential strain under one wheel of the dual load is larger than the other strain components. This selection of the strain component is generally not dealt with in the VESYS IIM program (1) or the Shell method (7), since only one wheel is used for strain computations. However, it is important for the summation applied in Miner's law for fatigue analysis to select only one component of the strain tensor.

In the computer program, the strain is computed either by interpretation of tabulated values read as data or by the regression formulas for tangential strain computations that are given below (Equations 6 and 7). The regression formulas for tangential strain under one wheel of the dual load at the bottom of the first and second layer of the three- or four-layer system are as follows:

$$\begin{aligned} E_4\epsilon_1/p = {} & 0.095\,929 - 0.039\,752 \log (T_1/a) - 0.025\,642 \log (E_1/E_4) \\ & - 0.068\,359 \log (E_2/E_4) - 0.008\,391 \log (E_3/E_4) \\ & - 0.020\,938\ [\log (T_1/a)]^2 + 0.020\,661 \log (T_1/a) \log (E_2/E_4) \\ & - 0.003\,075 \log (T_2/a) \log (E_1/E_4) - 0.010\,022 \log (T_2/a) \\ & \log (E_3/E_4) + 0.013\,842 \log (E_1/E_4) \log (E_2/E_4) \\ & + 0.009\,354\ [\log (E_2/E_4)]^2 + 0.003\,873 \log (E_2/E_4) \\ & \log (E_3/E_4) \end{aligned} \tag{5}$$

$R^2 = 0.83$

$$\begin{aligned} E_4\epsilon_2/p = {} & 0.085\,565 - 0.063\,772 \log (T_1/a) - 0.053\,237 \log (T_2/a) \\ & - 0.003\,243 \log [(1 + T_3)/a] - 0.021\,28 \log (E_1/E_4) \\ & - 0.018\,796 \log (E_2/E_4) - 0.066\,174 \log (E_3/E_4) \\ & + 0.029\,831 \log (T_1/a) \log (T_2/a) + 0.013\,190 \log (T_1/a) \\ & \log (E_2/E_4) + 0.037\,69 \log (T_2/a) \log (E_3/E_4) \\ & + 0.010\,535 \log (T_2/a) \log (E_1/E_4) + 0.003\,481 \log (T_2/a) \\ & \log (E_2/E_4) + 0.024\,568 \log (T_2/a) \log (E_3/E_4) \\ & + 0.004\,905 \log (E_1/E_4) \log (E_2/E_4) + 0.012\,278 \log (E_1/E_4) \\ & \log (E_2/E_4) - 0.002\,551\ [\log (E_2/E_4)]^2 + 0.017\,822 \\ & \log (E_2/E_4) \log (E_3/E_4) \end{aligned} \tag{6}$$

$R^2 = 0.84$

where

ϵ_1 = longitudinal (tangential) strain computed at the bottom of the upper asphaltic concrete layer under one wheel of the dual load;
ϵ_2 = longitudinal (tangential) strain computed at the bottom of the lower asphaltic concrete layer under one wheel of the dual load (applicable for an asphaltic concrete base layer or for an asphaltic overlay analysis);
p = contact pressure;

a = radius of contact;
T_1, T_2, T_3 = thicknesses of surface, base, and subbase layers, respectively; and
E_1, E_2, E_3, E_4 = elastic moduli of surface, base, subbase, and subgrade layers, respectively.

Note that the regression formulas were derived from the following values of variables: T_1/a = 0.4, 1.1, 2.2; T_2/a = 0.5, 1.3, 2.6; T_3/a = 0, 1.3, 2.6, 5.2; E_1/E_4 = 10, 60, 350; E_2/E_4 = 2, 5, 10, 60, 350; and E_3/E_4 = 1, 2, 5, 10.

Evaluation of Pavement Fatigue Life

The commonly used fatigue law that relates the applied tensile strain to the number of repetitions that cause failure is implemented in the system. The law reads

$$N_f = K_1\,(1/\epsilon)^{K_2} \tag{7}$$

where

N_f = number of strain repetitions that cause failure,
ϵ = applied strain, and
K_1, K_2 = regression coefficients from laboratory or field tests.

Correction factors for rest periods or residual stress effects could be incorporated at this stage into the K_1 and K_2 coefficients. The damage caused to the structure by wheel-load repetitions is evaluated by using Miner's law, i.e.,

$$D_j = \sum_{\ell=1}^{j} \sum_{i=1}^{k} (n_i/N_{fi}) \tag{8}$$

where

D_j = damage caused by k different load configurations during the previous j periods,
n_i = actual number of repetitions for load i, and
N_{fi} = number of repetitions to failure determined for load i by using Equation 7.

Evaluation of Cracked Area

The cracked area is evaluated by using the approach presented by Rauhut and others (4) and Kenis (1). The pavement geometry (layer thicknesses) and material properties (elastic moduli and fatigue law parameters) are treated stochastically, and the damage factor D_j is assumed to be a normally distributed random variable whose mean and variance can be computed by using Cornell's first-order, second-moment theory [Huffert and Lai (8)]. Under these assumptions, and for each D_j mean and variance, the probability F(1) that the variable D_j reaches the value of one (or the pavement is cracked) could be computed and used to evaluate the cracked area as follows:

$$\bar{C} = 1000\,[1 - F(1)] \tag{9}$$

where $\bar{C}$ is the expected cracked area (ft²/1000 ft²). It is used in Equation 1 or in Equation 4 to represent the cracked and patched area ($\bar{C} + \bar{P}$).

PERMANENT DEFORMATION SUBSYSTEM

The completed permanent deformation is based on the quasi-linear elastic analysis and incremental procedure proposed in three unpublished reports by J. Uzan, J.T. Christison, and K.O. Anderson [hereafter referred to as the Uzan reports (Rut Depth Prediction Using the Quasi-Elastic Approach; Prediction of Rut Depth Performance in Flexible Pavement Using Statistically Based Models; and Permanent Deformation in Asphalt Overlays on Flexible Pavements), which were done at Technion, Israel Institute of Technology, Haifa, 1979]. The procedure includes

1. Computation of the deflection bowl under loading and unloading conditions; the difference in deflection is the incremental (per repetition) residual surface deflection or rate of rutting;
2. Computation of the above rate of rutting at different stages during the pavement life; its integration enables estimation of the rut depth; and
3. For the quasi-linear elastic analysis, the moduli of elasticity under loading and unloading conditions are required.

It is shown that they are related to each other through the number of repetitions and the plastic parameters derived from repetitive loading tests.

In the actual permanent deformation, the incremental residual surface deflection is expressed as follows (from the Uzan reports):

$$d(RD)/dN = (pa/E_4) \cdot a_1\,N^{a_2} \tag{10}$$

and its integration over an interval of the pavement life (i.e., the contribution of the load repetition number between n_{j-1} and n_j) is

$$\Delta RD_j = (pa/E_4) \cdot [a_1/(1 + a_2)]\,(n_j^{1+a_2} - n_{j-1}^{1+a_2}) \tag{11}$$

where

RD = rut depth of permanent deflection,
ΔRD_j = increment of rut depth caused by a given load during period j of the pavement life,
p = contact pressure of the dual wheel,
a = contact radius of each wheel,
a_1, a_2 = regression coefficients [functions of the pavement geometry and material properties (elastic and plastic)], and
E_4 = subgrade elastic modulus.

It should be noted that Equation 10 is similar to the rutting model by Finn and others (2). According to both the models of Finn and others (2) and VESYS, there is some evidence that the permanent deformation is well correlated to the resilient deflection of the pavement. It was therefore introduced in the model through the regression coefficients a_1 and a_2.

Two different regression models were derived:

1. The first model assumes fixed transversal distribution of wheel loads and a standard deviation of 1 ft. By using the deflection bowl and the normalized distribution, the rut depth increment per repetition in Equation 10 is computed at two locations: (a) under one wheel of the dual load that corresponds to the maximum depression, and (b) at a 2-ft distance from the first position, on both the right and the left side, which corresponds to supports of the 4-ft straight edge. The contribution of the lateral distribution to the residual incremental deflection is taken proportional to the density function of the distribution. The difference between the residual deformation at the first location (under one wheel of the dual load) and the average of the residual deformations at the second locations (on the right and the left side) is the incremental rut depth. Note that this permanent deformation definition is in accordance with the

AASHO definition and procedure for measuring rut depth. Other program packages such as VESYS do not compute the rut depth but the absolute pavement deformation, as in our second model.

2. The second model computes the permanent deformation under one wheel of the dual load. The rut depth (as measured with a 4-ft straight edge) can be evaluated by computing the deformation under the wheel and at a 2-ft distance from the wheel and is the difference between these two computations.

The two approaches are implemented in the current computer program package. The regression formulas for a_1 and a_2 are given below (Equations 12-15). The use of either the first or second model at this stage is a matter of engineering judgment until the superposition law for permanent deformation is formulated and verified.

The regression formulas for the A_1 $(=1 + a_1)$ and A_2 $(=1 + a_2)$ used in Equations 10 and 11 are given below:

1. The first case that corresponds to a given transversal distribution of wheel loads is as follows:

$$\begin{aligned}\log A_1 = {} & 0.0281 \log\alpha_2 + 0.0916 \log\alpha_3 - 0.0251 \log\alpha_4 + 0.0365 \\ & \log(1+\mu_1) + 0.1051 \log(1+\mu_2) + 0.0544 \log(1+\mu_3) \\ & + 0.0705 \log(1+\mu_4) + \log W[0.009 \log\alpha_2 + 0.0894 \log\alpha_3 \\ & - 0.039 \log\alpha_4 + 0.0425 \log(1+\mu_1) + 0.054 \log(1+\mu_2) \\ & + 0.075 \log(1+\mu_3) + 0.1025 \log(1+\mu_4)] + 0.0122 \log\alpha_1 \\ & \log\alpha_2 + 0.0255 \log\alpha_1 \log(1+\mu_2) + 0.013 \log\alpha_2 \log(1+\mu_1) \\ & - 0.1502 \log\alpha_2 \log(1+\mu_2) - 0.1752 \log\alpha_3 \log(1+\mu_1) \\ & - 0.3116 [\log(1+\mu_2)]^2 \end{aligned} \tag{12}$$

$R^2 = 0.96$

$$\begin{aligned}A_2 = {} & 0.054 + 0.0093 \log W - 1.424 \log\alpha_1 - 0.457 \log\alpha_2 + 1.428 \log\alpha_3 \\ & + 0.0265 \log\alpha_4 + 0.217 \log(1+\mu_1) - 0.4523 \log(1+\mu_2) \\ & - 0.183 \log(1+\mu_3) + 2.428 \log W \log\alpha_3 - 3.3946 \log\alpha_1 \log\alpha_2 \\ & + 1.442 (\log\alpha_2)^2 - 2.237 \log\alpha_2 \log(1+\mu_2) \end{aligned} \tag{13}$$

$R^2 = 0.94$

2. The second case is for no transversal distribution of wheel loads:

$$\begin{aligned}\log A_1 = {} & 0.0266 \log\alpha_2 + 0.055 \log(1+\mu_1) + 0.1593 \log(1+\mu_2) \\ & - 0.1084 \log(1+\mu_3) + 0.7798 \log(1+\mu_4) \\ & + \log W [0.0260 \log\alpha_2 + 0.1662 \log\alpha_4 + 0.0281 \log(1+\mu_1) \\ & + 0.1872 \log(1+\mu_2) + 0.2132 \log(1+\mu_3) + 0.8103 \log(1+\mu_4)] \\ & + 0.0293 \log\alpha_1 \log\alpha_2 + 0.0757 \log\alpha_1 \log(1+\mu_3) \\ & - 1.6121 (\log\alpha_4)^2 - 0.1072 \log\alpha_4 \log(1+\mu_2) + 0.2018 \log\alpha_4 \\ & \log(1+\mu_3) + 0.0508 [\log(1+\mu_1)]^2 + 0.0860 \log(1+\mu_1) \\ & \log(1+\mu_3) + 2.4116 [\log(1+\mu_3)]^2 \end{aligned} \tag{14}$$

$R^2 = 0.97$

$$\begin{aligned}A_2 = {} & 0.0884 - 1.1007 \log\alpha_1 - 0.9640 \log\alpha_2 - 0.2455 \log(1+\mu_2) \\ & - 1.4252 \log(1+\mu_3) + 0.6959 (\log\alpha_1)^2 - 3.220 \log\alpha_1 \log\alpha_2 \\ & - 0.9517 \log\alpha_1 \log(1+\mu_1) + 1.2965 \log\alpha_1 \log(1+\mu_2) \\ & + 6.0467 \log\alpha_1 \log(1+\mu_4) + 0.6490 (\log\alpha_2)^2 - 0.3772 \log\alpha_2 \\ & \log(1+\mu_3) + 8.0917 \log(1+\mu_2) \log(1+\mu_3) + 7.4075 \log(1+\mu_3) \\ & \log(1+\mu_4) \end{aligned} \tag{15}$$

$R^2 = 0.95$

where

$W = wE_4/pa$ = nondimensional deflection;
w = elastic deflection between wheels of the dual load;
p = contact pressure;
a = radius of contact;
T_1, T_2, T_3 = thicknesses of surface, base, and subbase layers, respectively;
E_1, E_2, E_3, E_4 = elastic moduli of surface, subbase, and subgrade layers, respectively;
$\alpha_1, \alpha_2, \alpha_3, \alpha_4$ = alpha's plastic material properties of layers; and
$\mu_1, \mu_2, \mu_3, \mu_4$ = mu's plastic material properties of layers.

The nondimensional deflection (W) was expressed as a function of the four-layer system variables as follows:

$$\begin{aligned}\log W = {} & 0.2847 - 0.2361 \log(T_1/a) - 0.0898 \log(T_2/a) \\ & - 0.2441 \log(E_1/E_4) - 0.1830 \log(E_2/E_4) - 0.0460 \log(E_3/E_4) \\ & + \log(T_1/a)\{-0.2075 \log(T_1/a) + 0.2015 \log(T_2/a) \\ & + 0.0986 \log[(1 + T_3)/a] - 0.2191 \log(E_1/E_4) \\ & + 0.1246 \log(E_2/E_4) + 0.1020 \log(E_3/E_4)\} + \log(T_2/a) \\ & \{-0.1720 \log(T_2/a) + 0.1143 \log[(1 + T_3)/a] + 0.0118 \\ & \log(E_1/E_4) - 0.1919 \log(E_2/E_4) - 0.0965 \log(E_3/E_4)\} \\ & + \log[(1 + T_3)/a] [-0.0474 \log(E_1/E_4) + 0.0220 \log(E_2/E_4) \\ & + 0.0220 \log(E_3/E_4)] + \log(E_1/E_4) [-0.3315 \log(E_1/E_4) \\ & + 0.0145 \log(E_2/E_4) + 0.0095 \log(E_3/E_4)] + \log(E_2/E_4) \\ & [0.0216 \log(E_2/E_4) + 0.0074 \log(E_3/E_4)] \\ & + 0.0300 [\log(E_3/E_4)]^2 \end{aligned} \tag{16}$$

The rut depth variance, as required for serviceability evaluation as per Equation 4, is computed by using Cornell's first-order, second-moment theory (8) for the functions that are represented by Equation 11 and the regression formulas (Equations 12-16). It includes the effect of variation of pavement geometry (layer thicknesses) and of material properties (resilient moduli and plastic parameters of all layers).

PROGRAM DESCRIPTION

The system that has been developed to predict pavement performance includes several elements and models for describing the structure, the different materials, and the environmental and loading conditions. They are presented and discussed in the following sections.

Pavement Design Framework

The pavement design framework consists of a four-layer system with an option for selecting only the three-layer system. This is especially convenient for overlay analysis. In its early life, the three-layer system is for describing the conventional pavement while the four-layer system is used for the overlaid pavement.

The material properties required for structural analysis are the linear-elastic moduli, which are assumed temperature and/or suction dependent. Strain and rut depth computations are made by using the regression formulas (Equations 5 and 6 and 12-16).

Loads

Loads are represented by dual wheels with constant contact radius (a = 4.5 in) and constant distance between wheels (c - c = 3a). Loads of varying intensities can be prescribed. Their contributions to fatigue life or cracking are summed by using Miner's law. Note that the sequential order of application of the loads does not affect the results. However, it does affect the rate of permanent deformation, as seen from Equations 10 or 11. In this case, the time-hardening scheme (9) is adopted for load equivalency computations that are made each month during the pavement life.

Environmental Conditions

The environmental conditions include temperature distribution of each asphalt concrete layer and temperature and suction (or moisture content) distribution of the subgrade. The analysis, and therefore the input, is made for each basic time period that has been chosen to be a monthly interval. For the asphalt concrete modulus of elasticity, which strongly depends on temperature, the following three-segment relation is assumed:

$$\text{When } T \leqslant 32°\text{F}, E_T = E_o \tag{17}$$

$$\text{When } 32°\text{F} < T \leqslant 77°\text{F}, E_T = E_o\ 10^{(T-32)\alpha_1} \tag{18}$$

$$\text{When } T < 77°\text{F}, E_T = E_o\ 10^{[45\alpha_1+(T-77)\alpha_2]} \tag{19}$$

where

E_T = modulus of elasticity of asphalt concrete at temperature T (°F),
E_o = E_T at T = 32°F = 0°C, and
α_1, α_2 = material coefficients.

The temperature and suction (or moisture content) dependence of the modulus of elasticity of the subgrade are assumed as follows (10):

$$E_s = E_{sr} \cdot a_h \cdot a_T \tag{20}$$

$$a_h = \exp[a_5(h - h_s)] \tag{21}$$

$$a_T = \exp\{a_3(T - T_s)/[a_4 + (T - T_s)]\} \tag{22}$$

and

$$\ln h = a_1 - a_2 \ln w \tag{23}$$

where

E_s = subgrade modulus at temperature T and suction h or moisture content w,
E_{sr} = subgrade modulus at given reference temperature T_s and given reference suction h_s,
a_h = suction influence factor,
a_T = temperature influence factor, and
a_1,a_2,a_3,a_4,a_5 = constants.

Note that when the subgrade temperature is 32°F or less, the material is assumed to be frozen and a representative modulus of elasticity of 50 000 psi is assigned to it (2). During the frozen period, which should be specified, no rut depth computations are made, since no contribution could be made during these months.

The environmental conditions indirectly affect the material properties of granular subbase and base layers through the dependence of their moduli of elasticity on that of the subgrade. Two ratios of granular material and subgrade moduli are implemented: the Shell method (7) and the USCE method (11).

Overlay Analysis

The overlay analysis is included in the program for maintenance strategy studies. In this case, cracking involved in Equation 4 is the one that is computed in the upper layer (overlay layer). Rut depth and its variance are initialized (to zero and to the corresponding initial serviceability) and new plastic parameters (alphas and mus) of the existing pavement layers are specified. This makes it possible to take into account the changes in material properties that result from past traffic conditions. The computations are then made for the renewed pavement, and its performance is predicted.

Note that the period preceding overlay application can be subdivided into subperiods that are multiples of a year. This facility enables the designer to include changes in material properties during the pavement life that are caused by environmental and/or loading conditions (such as for cracked asphalt or any special conditions).

APPLICATION

System Description

Application of the procedure is illustrated by analyzing the pavement response of an AASHO Road Test section. The analysis presented below includes a sensitivity analysis of different pavement response components (cracking, rut depth, and its variance) to input parameter variations. The section chosen is section 581 of loop 4 in the AASHO Road Test. The input variables were collected from the literature and are summarized below:

1. Geometry: The geometry consists of 5-in asphalt concrete, 6-in base course, and 13-in subbase course on the top of the silty clay subgrade.
2. Moduli of elasticity: The asphalt concrete material is temperature dependent according to Equations 17-19, where $E_o = 1.8 \times 10^6$ psi, $\alpha_1 = 0.014$, and $\alpha_2 = 0.022$, which lead to elastic moduli as reported by Finn and others (2). The subgrade modulus is temperature and moisture dependent according to Equations 20-23, where E_{sr} = 6000 psi, $a_1 = 0$, $a_2 = -1$, $a_3 = -0.070$, $a_4 = 33.50$, and $a_5 = -0.70$, which were chosen to reduce the elastic modulus by half in the springtime period. The granular material modulus was computed by using the procedure by Barker and others (11).
3. Plastic properties: The following alphas and mus were chosen on the basis of the literature review (4,5, and Uzan reports): $\alpha_1 = 0.56$, $\mu_1 = 0.3$, $\alpha_2 = \alpha_3 = 0.9$, $\mu_2 = \mu_3 = 0.3$, $\alpha_4 = 0.80$, and $\mu_4 = 0.045$.
4. Fatigue parameters: The fatigue law from Finn and others (2) for 10 percent cracking and a corrected one (K_1 = 0.057 943 $E^{-0.854}$ and 0.3 $E^{-0.854}$, K_2 = 3.291) are implemented.
5. Environmental conditions: The asphalt temperature distribution was 48, 37, 32, 31, 35, 62, 73, 82, 87, 88, 78, and 63 for the 12 months following the onset of the test (repeated for the next 12 months). The subgrade was assumed to be frozen during the period between the second and fifth month and thawed during the sixth and seventh month.
6. Loading: The traffic is comprised of 18-kip axle loads with an assumed standard deviation of the wander of 1 ft. The traffic was not uniformly distributed throughout the test period. The actual number of repetitions was taken from the AASHO Road Test data (3).
7. Material variability: It was assumed that

Figure 1. Results of analysis of AASHO Road Test section.

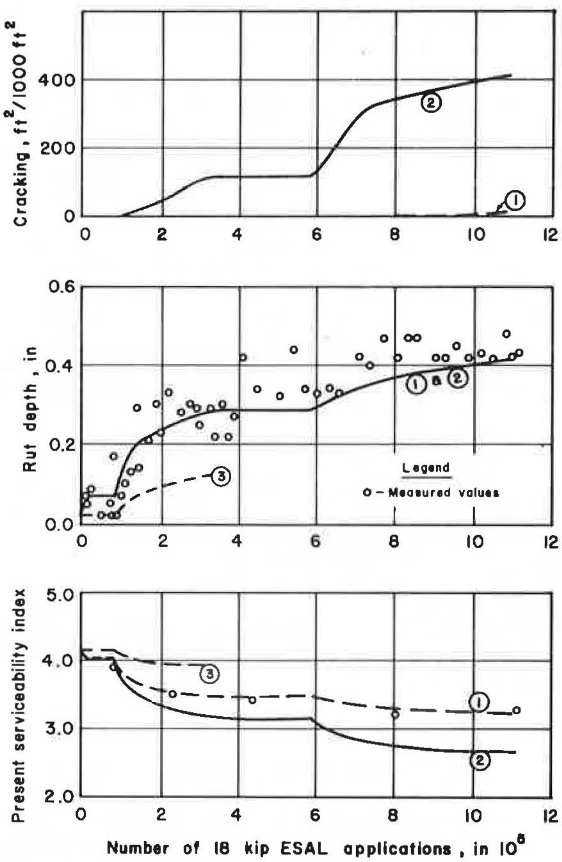

Figure 2. Results of sensitivity analysis and variation of means.

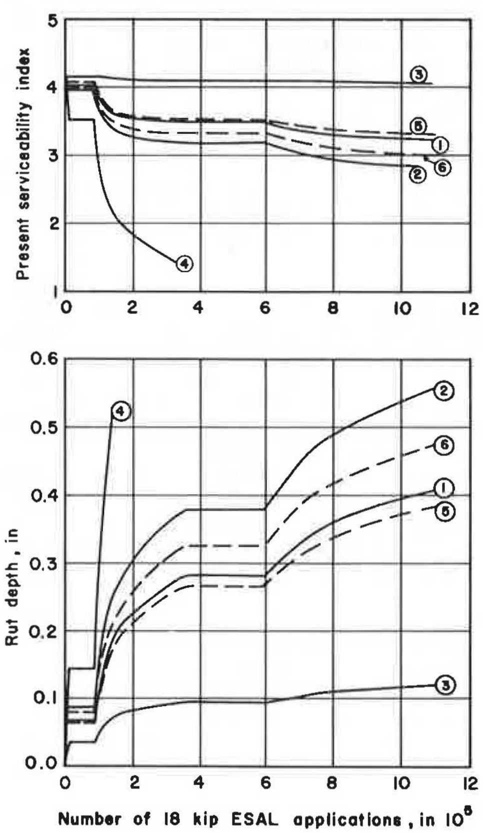

Figure 3. Results of sensitivity analysis and varying the coefficients of variation.

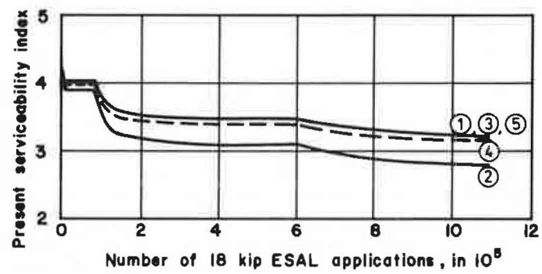

Table 1. Variable values that correspond to different curves in Figure 2.

Curve No.	α_1	μ_1	α_4	E_4 (psi)
1	0.56	0.30	0.80	6000
2	0.56	0.50	0.80	6000
3	0.70	0.30	0.80	6000
4	0.45	0.30	0.80	6000
5	0.56	0.30	0.95	6000
6	0.56	0.30	0.80	5000

the coefficient of variation of layer thicknesses, moduli of elasticity, and plastic parameters is 0.10. The regression coefficients were assumed nil for those input variables except for the subbase, base, and subgrade moduli that were fully dependent. As for the fatigue law, the coefficients of variation of the K_1 and K_2 variables were 0.3 and their regression coefficient was 0.857 (4,12).

Results

The results of the analysis are presented in Figures 1, 2, and 3, which describe the pavement performance (serviceability index, rut depth, and cracking area versus number of repetitions).

Figure 1 shows that about a 50 percent cracked area was obtained at the end of the test period when the fatigue law derived by Finn and others (2) is implemented (curve 2). Because the section did not crack, it is concluded that the fatigue law derived by Finn and others (2) in conjunction with a nonlinear computation scheme cannot be implemented in the program. The K_1 parameter was therefore corrected to correspond to minor cracking (curve 1). Figure 1 also shows the predicted performance of the pavement for the above two cases. Because the only difference is in the cracked area, the serviceability index computed by using the fatigue law (2) is lower than the index computed by using the corrected fatigue law. Measured rut depths and performance are reproduced in Figure 1 to evaluate the reliability of the prediction for the chosen input variables. It is seen that the predicted and measured shape and values are in very good agreement.

Sensitivity Analysis

The fatigue subsystem is not included in this analysis, since the section did not fail at the end of the test period. This will simplify the analysis of the rut depth and of its variance (and through these variables, the serviceability index). Moreover, we feel that the fatigue subsystem should be complemented to take into account the rest period, crack propagation, crack retardation, etc. Until then, it appears that the cracking is yet unpredictable, leading to the introduction of correction factors such as that proposed by Brown (13).

Results of the sensitivity analysis of the rut depth are presented in Figures 1 and 2, which illustrate the effect of varying only the variables that are of significance, such as α_1, μ_1, α_4, and E_4 (see Table 1). Figure 2 shows that the ranking of these variables by their effect on rut depth is similar to that reported by Rauhut and others (4,5) in their sensitivity analysis that used VESYS II. Figure 2 also shows the effect of varying these variables on the serviceability index. It is seen that the main effect is due to α_1 through its influence on the rut depth. Figure 1 illustrates the effect of the plastic properties of the base, subbase, and subgrade from the above values to perfectly elastic properties that are assumed to correspond to the overlay case (curve 3 in Figure 1). It is seen that the rut depth is reduced to less than half its value and the serviceability index remains high. This behavior corresponds with experience with overlay or step construction (Uzan report, Permanent Deformation in Asphalt Overlays on Flexible Pavements).

The rut depth variance is sensitive not only to the mean values of the input variables but also to their variance. According to Rauhut and others (4,5), the slope variance, which was replaced by the rut depth variance in Equation 3, is affected by almost the same variable variations as was the rut depth. However, this was expected to happen, since the slope variance was expressed as a function of the rut depth (Equation 2). Because this does not correspond to the current case, the sensitivity analysis is conducted only for the effect of variation of the coefficient of variation of α_1, μ_1, E_1, and E_4. Figure 3 shows the effect of varying the coefficient of variation of only one variable. Curve 1 represents the performance of the section for a 0.10 coefficient of variation; other curves correspond to an α_1's coefficient of variation of 0.2 for curve 2, a μ_1's coefficient of 0.2 for curve 3 (superimposed with curve 1), and an E_4's coefficient of 0.27 for curve 5 (also superimposed with curve 1). It is seen that the main effect is again due to varying the coefficient of variation of α_1.

From the results shown in Figures 2 and 3, it can be concluded that the pavement performance is highly affected by the α_1 plastic property.

DISCUSSION OF RESULTS

The results of the analysis presented refer to a particular section of the AASHO Road Test. The loss in serviceability in this section was mainly due to rutting and slope variance. It is seen that the fatigue law derived by Finn and others (2) from AASHO Road Test data and the PSAD program are not applicable in the present program. (This fatigue law seems appropriate only in the framework of the PDMAP program.) Because no other fatigue law is available and no cracking developed in the section, the fatigue law was therefore adjusted to correspond to minor cracking. However, note that a basic approach for fatigue analysis (based on fracture mechanics) is warranted to release from correction factors of the order of 13-18 (2) or 100 (13).

The computed rut depth versus number of repetitions is very similar to the measured one. Note that the rut depth is very sensitive to the plastic material properties of all layers, especially the upper asphaltic one. Similar results could be obtained with a different set of plastic properties. Because these properties are of primordial importance in the rut depth evaluation, their values and dependence on material and stressing variables should be investigated further. In addition, the rut depth variance was found to be primarily affected by the alpha of the upper asphaltic layer and its variation. Therefore, since there is an indication of the stress and temperature dependence of this parameter, it should be investigated carefully at stressing conditions similar to field conditions. The model for rut depth prediction is in essence similar to that developed by Uzan and others (Rut Depth Prediction Using Quasi-Elastic Approach) extended to simple statistically based models, and verified by using test sections from AASHO Road Test and Alberta and Ontario Brampton Test Road (Uzan reports, Prediction of Rut Depth Performance in Flexible Pavement Using Statistically Based Models; Permanent Deformation in Asphalt Overlays on Flexible Pavements). The results were very satisfactory. The current program is therefore expected to bring similar good results. However, information concerning material properties and performance as required by the program was not available to us.

The pavement performance predicted by the program is in very good correlation with the measured one. Note that the curvature of the serviceability index versus the number of repetitions is rather upward (sag curvature). This results from the special combination of the traffic distribution and environmental conditions.

In conclusion, the program package seems reliable and could be widely and easily implemented and programmed on a microcomputer.

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Structural Analysis of AASHO Road Test Flexible Pavements for Performance Evaluation

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The structural analysis of American Association of State Highway Officials (AASHO) Road Test flexible pavements was performed for the specific purpose of developing a pavement performance model that would be implemented in a pavement management system used by the U.S. Forest Service. For this reason, a precise and highly sophisticated structural evaluation was not made. However, it was determined that the nonlinear elastic properties of unbound pavement materials and seasonal material conditions should be characterized. The use of a thin layer BISAR elastic-layer analysis helped to overcome difficulties in the structural analysis associated with the PSAD2A elastic-layer program. An equivalent layer procedure was found to give results similar to the thin layer BISAR analysis. A modified BISAR program was developed that incorporated the equivalent layer procedure and was used in the structural analysis of the AASHO Road Test pavements. Results of the analysis showed that predicted pavement deflections from the structural analysis compared very well with spring and fall deflection measurements taken at the AASHO Road Test. An evaluation of the modulus ratios of adjacent unbound pavement layers led to the conclusion that the modulus ratios are not fixed within a narrow range of values but can vary significantly depending on the state of stress in the pavement layers.

The structural analysis of flexible pavements can involve a wide range of methodologies, which range from sophisticated finite-element modeling that considers nonlinear elastic and viscoelastic properties of pavement materials to relatively uncomplicated elastic-layer techniques that have various simplifying assumptions regarding material properties, loading conditions, etc. Therefore, it is important to choose the appropriate level of sophistication for the particular situation being analyzed.

This paper describes a structural analysis of American Association of State Highway Officials (AASHO) Road Test flexible pavement sections, which was conducted as part of a cooperative research effort by the U.S. Forest Service and the University of Texas at Austin. The objective of the analysis was to calculate pavement response parameters that could be compared with AASHO Road Test pavement performance data. From this information, a pavement performance model would be developed and used to revise and improve an existing pavement management system (1).

The structural analysis in this study was not a precise and highly sophisticated evaluation of the pavement structures. Because of the number of AASHO Road Test sections to be studied, practical restrictions were necessary in the consideration of computer execution time. In addition, since the pavement performance model being developed would be included in a pavement management system, it could be assumed that a similar structural analysis would have to be employed in that system. For these reasons, a relatively simple analysis was favored. However, because of important economic comparisons made among candidate pavement materials in the pavement management system, it was felt necessary to consider the stress-sensitive properties of unbound pavement materials. Because pavement performance varies considerably, depending on climatic and seasonal conditions, it was also determined that the characterization of seasonal material properties was important in the structural analysis. These factors tended to indicate that a more sophisticated evaluation was necessary.

The following sections of this paper discuss the evaluation of different methodologies for performing the analysis. Results from this evaluation are discussed, including results from the structural analysis. A comparison is made between measured and predicted pavement deflections, and the modulus ratios for adjacent unbound pavement layers are studied.

PROCEDURE FOR NONLINEAR ELASTIC ANALYSIS

In the structural analysis of AASHO Road Test pavement sections, the asphalt layer was assumed to have linear elastic properties, whereas the granular base and subbase materials and the fine-grained subgrade were assumed to have nonlinear stress-dependent characteristics. The stress-sensitive nature of the unbound pavement materials was characterized by the following relations. For fine-grained materials,

$$M_R = A\,\sigma_d^B \tag{1}$$

where

M_R = resilient modulus of fine-grained material,
σ_d = principal stress differences ($\sigma_1 - \sigma_3$) or deviator stress (psi), and
A, B = experimentally determined coefficients that define the behavior of the fine-grained material.