Predicting Remaining Strength of Flexible Pavement and Overlay Design Thickness with Stochastic Modeling

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2 authors, including:

Khaled Abaza

Birzeit University

Predicting Remaining Strength of Flexible Pavement and Overlay Design Thickness with Stochastic Modeling

Khaled A. Abaza and Maher M. Murad

A pavement's remaining strength is predicted from initial pavement strength by means of layer structural capacity adjustment factors. "Initial pavement strength" is defined as the total structural capacity associated with the asphalt concrete and underlying granular layers estimated by appropriate relative strength indicators such as the gravel equivalent or structural number. The pavement's future remaining strength is estimated as the product sum of multiplying the initial layer strength by the corresponding adjustment factor. The structural adjustment factor represents the percentage of remaining strength for a particular pavement layer at a specified service time (*t*). The structural capacity adjustment factor associated with the asphalt layer is the principal factor estimated from a project performance curve generated with stochastic modeling. The discrete time Markov model is used to predict pavement distress ratings for a particular project over a specified service life (*T*). A simplified approach is presented for estimating a project transition matrix using only initial and terminal transition probabilities. The predicted distress ratings are used to construct a project performance curve. The area falling under the performance curve has long been recognized as a direct measure of the pavement relative strength. Therefore, the principal structural capacity adjustment factor is defined as the ratio of the area under the performance curve for the remaining service period ($T - t$) and the area under the entire performance curve for a service life (*T*). A sample application is provided with results used to generate empirical models to aid in developing rehabilitation strategies and management policies at the network level.

Prediction of flexible pavement remaining strength has potential applications in pavement rehabilitation and management. Current methods for estimating flexible pavement remaining strength are either mechanistic, requiring measurement of surface deflection, or empirical and based on an assessment of pavement distress. The first method requires instruments such as the Dynaflect or falling weight deflectometer with results used in a backcalculation of the multilayered linear elastic theory (*1–3*). The second method, known as the effective thickness approach or component analysis method, requires assessing pavement distress with the outcome translated into equivalency conversion factors (*4–6*). These methods mainly yield a present estimate of the pavement remaining strength for calculating the required overlay thickness at the project level. The ability to predict the future pavement remaining strength can be potentially useful in developing long-term rehabilitation strategies and management policies at the network level.

Prediction of pavement remaining strength is directly related to the long-term performance of pavements. Two types of models have been widely used in predicting the long-term performance of pavements: probabilistic and deterministic. The probabilistic model predicts the future pavement condition with some degree of uncertainty, whereas the deterministic model predicts it with certainty (*7*). The probabilistic model that was extensively used by several researchers to predict pavement performance is the discrete time Markov model (*8–12*). The Markov model can be used with homogenous or nonhomogenous transition chains. The homogenous chains require the same transition matrix, whereas the nonhomogenous chains can deploy a different transition matrix for each chain.

The prediction outcome of pavement performance is typically represented by means of a performance curve, which depicts pavement deterioration rates in relation to service time. Predicted performance curves were used in yielding optimum pavement design and performing optimum life-cycle analysis (*13, 14*). The area falling under the performance curve has long been recognized as a direct measure of the pavement relative structural capacity (*4, 5, 13, 14*). Therefore, the area under the performance curve at any given service time can be directly related to the pavement remaining strength, considering a specified service life. It is proposed to use the discrete-time Markov model to predict pavement distress ratings for use in generating distinct performance curves (models). The predicted performance models will be used in estimating the pavement remaining structural capacity. The predicted remaining structural capacity can be used in estimating the required overlay design thickness, developing pavement rehabilitation strategies, and establishing rehabilitation project priority scheduling.

RESEARCH OBJECTIVES

There are three main objectives for this research paper:

1. Predicting the performance curve (model) for a particular pavement project using stochastic modeling;
2. Predicting the flexible pavement remaining strength and overlay design thickness using layer structural capacity adjustment factors with the principal asphalt layer adjustment factor derived from the generated performance curve; and
3. Developing empirical stochastic-based models for predicting remaining strength and overlay design thickness to be used by the practitioners in pavement rehabilitation and management.

K. A. Abaza, Department of Civil Engineering, Birzeit University, P.O. Box 14, Birzeit, West Bank, Palestine. M. M. Murad, Department of Civil Engineering Technology, University of Pittsburgh at Johnstown, 450 Schoolhouse Road, Johnstown, PA 15904. Corresponding author: K. A. Abaza, kabaza@birzeit.edu.

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METHODOLOGICAL DEVELOPMENT

Methodological development includes three main sections. The first section presents the stochastic model that was previously used to predict pavement performance. The second section presents a newly developed simplified approach for estimating the transition matrix using only two or three transition probabilities. This approach substantially reduces the time and effort involved in estimating what are, otherwise, a larger number of transition probabilities required to apply the Markov model. The third section presents a new approach developed for predicting flexible pavement remaining strength using performance curves as the main input parameter. Currently used methods only estimate the present remaining strength and require the use of expensive structural pavement testing devices. Predicting the future remaining strength, as presented in this paper, can be very beneficial in pavement management applications that generally require low-cost distress assessment procedures.

Stochastic Model for Predicting Pavement Performance

The stochastic model that was extensively used in predicting future pavement conditions is the Markov model (*8–12*). The basic Markov model for discrete-time homogenous chains is presented in Equation 1. The model predicts a column vector of state probabilities [$Q^{(k)}$] after a period comprising k discrete-time intervals (transitions) from multiplying the row vector of initial state probabilities [$Q^{(0)}$] by the transition matrix [$P^{(k)}$] multiplied k times. The transition matrix used in Equation 1 contains the transition probabilities that remain unchanged if homogenous chains are assumed. The initial state probabilities for new pavements can be assumed to have values as defined in Equation 1. This condition can be met if a reasonably large number of pavement states is used. A transition matrix with 10 states is generally adequate to ensure that this condition is satisfied. Otherwise, an initial pavement distress assessment is required to estimate the initial state probabilities.

$$Q^{(k)} = Q^{(0)}P^{(k)} \qquad (k = 1, 2, \ldots, n) \tag{1}$$

where

$\sum_{i=1}^{m} Q_i^{(k)}$ = 1.0 (k = 1, 2, . . . , n);

$Q^{(0)}$ = ($Q_1^0, Q_2^0, Q_3^0, \ldots, Q_m^0$), the row vector representing initial state probabilities, [(1, 0, 0, . . . , 0) for new pavements]; and

$Q^{(k)}$ = column vector representing state probabilities after k transitions.

The homogenous transition matrix used in estimating the future state probabilities in the absence of any maintenance and rehabilitation (M&R) works is defined by Equation 2. The transition matrix is a square matrix with size (m) representing the number of deployed pavement states. Each row of the transition matrix is typically assumed to only include the two transition probabilities ($P_{i,i}$) and ($P_{i,i+1}$) (*8, 9, 11, 12*). The transition probabilities along the main matrix diagonal ($P_{i,i}$) represent the probabilities that pavements currently in condition state (i) will remain in the same condition state after the elapse of one transition. The transition probabilities ($P_{i,i+1}$) represent pavement deterioration rates from a present condition state (i) to a worse state ($i + 1$) after one transition. All matrix entries below the main diagonal represent pavement improvement rates, which are assigned zero values in the absence of M&R works. The main objective in defining the transition matrix, as presented in Equation 2, is to predict the future performance of new pavements.

$$P = \begin{pmatrix} P_{1,1} & P_{1,2} & 0 & 0 & 0 & \cdots & 0 \\ 0 & P_{2,2} & P_{2,3} & 0 & 0 & \ldots & 0 \\ 0 & 0 & P_{3,3} & P_{3,4} & 0 & \ldots & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & & \vdots \\ 0 & 0 & 0 & 0 & P_{m-1,m-1} & & P_{m-1,m} \\ 0 & 0 & 0 & 0 & 0 & & P_{m,m} \end{pmatrix} \tag{2}$$

where $P_{i,i} + P_{i,i+1}$ equals 1.0 and $P_{m,m}$ equals 1.0.

The future pavement distress rating [$DR^{(k)}$] for a particular pavement project can be estimated as defined in Equation 3. The future state probabilities [$Q_i^{(k)}$], as determined from the Markov model, are used as the main parameters for predicting the future pavement distress ratings. The pavement distress rating after k transitions [$DR^{(k)}$] is estimated as the mean of a compound uniform probability density function defined by the future state probabilities [$Q_i^{(k)}$]. The ***i***th future state probability represents a uniform probability density function, with its ordinate represented by [$Q_i^{(k)}$], and its random variable range defined using the lower (LDR_i) and upper (UDR_i), distress ratings. The mean distress rating (B_i) is defined as the average of the lower and upper distress ratings used to define the pavement condition state (i) according to a deployed pavement distress indicator.

$$DR^{(k)} = \sum_{i=1}^{m} B_i Q_i^{(k)} \qquad (k = 0, 1, 2, \ldots, n) \tag{3}$$

where

$$B_i = \frac{LDR_i + UDR_i}{2}$$

and

$$B_m \le DR^{(k)} \le B_1.$$

The predicted distress ratings [$DR^{(k)}$] can be used to construct a distinct performance curve for a particular pavement project, as shown in Figure 1. The predicted distress ratings are plotted against the corresponding number of transitions (k) as shown, or by using the equivalent service time (t) in years obtained from multiplying the number of transitions (k) by the time interval length (d) in years. The length of time interval (transition) is typically taken to be equal to 1 or 2 years.

Simplified Stochastic Approach for Predicting Pavement Performance

The major requirement in predicting future distress ratings for a particular pavement project is the estimation of the corresponding transition matrix. The transition matrix, as defined in Equation 2, requires obtaining estimates of (m − 1) transition probabilities. Estimation of (m − 1) transition probabilities minimally requires conducting two cycles of pavement distress assessments, separated by a time period equal to the length of time interval (d) equivalent to the length of one transition (*11, 12*). This would require extensive efforts, especially if the transition matrix must be estimated for individual pavement projects. For estimating the transition matrix, this section presents a simplified approach that only requires the use

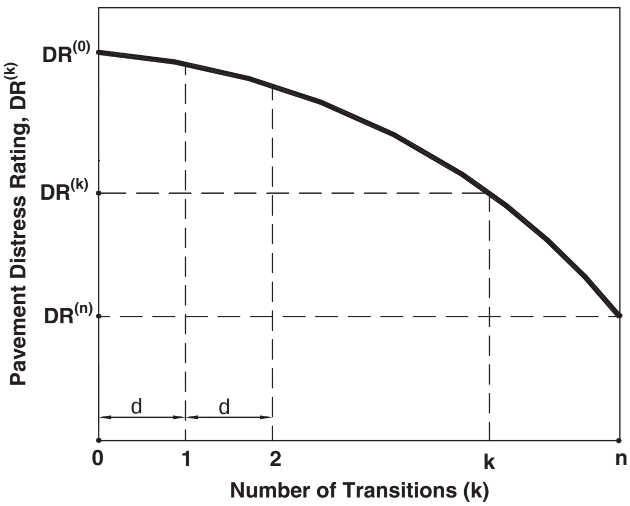

FIGURE 1 Pavement distress rating predicted with stochastic modeling.

of two or three transition probabilities. The simplified approach is based on the typical pavement deterioration trends that are recognized in pavement performance. The typical performance curve is either represented by a straight line, indicating uniform deterioration rates; a polynomial concaving upward, indicating increasing deterioration rates; or a polynomial concaving downward, indicating decreasing deterioration rates (*4, 14*). These three typical trends of pavement deterioration can be related to the transition probabilities to be used in generating a particular project performance curve.

The proposed simplified approach for estimating the transition matrix requires only three deterioration rates. These three deterioration rates include initial ($P_{1,2}$), middle ($P_{x,x+1}$), and terminal ($P_{m-1,m}$), transition probabilities. Therefore, three deterioration rates are to be estimated from pavement distress assessment conducted on pavements in condition states (1), ($m-1$), and the condition state (x), which is selected to be the middle state between states (1) and ($m-1$). Estimates of these three deterioration rates are required to approximate the remaining deterioration rates ($P_{i,i+1}$) under the assumption of either uniformly increasing or uniformly decreasing deterioration rates. Generally, there are four distinct performance prediction models that can be derived from the transition matrix, depending on the relationship among the deployed transition probabilities, as described below.

Increasing Rate of Deterioration Model

The first performance prediction model is represented by a polynomial of second degree, as indicated by Equation 4. The model for pavement distress rating [DR(t)] at a service time t is obtained from fitting a second-degree polynomial to the predicted performance curve shown in Figure 1. The second derivative for this model is negative, indicating that the corresponding performance curve is concave upward. The model coefficients a, b, and c are constants obtained from curve-fitting the predicted distress ratings [$\text{DR}^{(k)}$] using either best-fitting or regression techniques.

$$\text{DR}(t) = -at^2 - bt + c \tag{4}$$

The performance prediction model indicated by Equation 4 is associated with progressively increasing deterioration rates. This deterioration trend takes place when the initial deterioration rate ($P_{1,2}$) is smaller than the middle deterioration rate ($P_{x,x+1}$), which is in turn smaller than the terminal deterioration rate ($P_{m-1,m}$). It is not too unrealistic to assume that the deterioration rates associated with this model increase uniformly, especially given that the corresponding performance curve is parabolic in nature with consistently increasing slopes. Therefore, the deterioration rates ($P_{i,i+1}$) are determined using Equation 5a for condition states falling between (2) and ($x-1$), on the basis of the assumption that deterioration rates uniformly increase from an initial value of ($P_{1,2}$) to a terminal value of ($P_{x,x+1}$). Similarly, the deterioration rates associated with condition states falling between ($x+1$) and ($m-2$) are determined using Equation 5b, assuming that deterioration rates uniformly increase from an initial value of ($P_{x,x+1}$) to a terminal value of ($P_{m-1,m}$).

$$P_{i,i+1} = P_{1,2} + (i-1)\left(\frac{P_{x,x+1} - P_{1,2}}{x-1}\right) \qquad (i = 2, 3, \ldots, x-1) \tag{5a}$$

$$P_{i,i+1} = P_{x,x+1} + (i-x)\left(\frac{P_{m-1,m} - P_{x,x+1}}{m-x-1}\right) \qquad (i = x+1, x+2, \ldots, m-2) \tag{5b}$$

where

$$P_{1,2} < P_{2,3} < \cdots < P_{x,x+1} < P_{x+1,x+2} < \cdots < P_{m-1,m}$$

Uniform Rate of Deterioration Model

The second type of deterioration model is associated with a uniform rate of deterioration as presented in Equation 6. The corresponding prediction model takes on a linear form, with coefficients b and c determined from the predicted distress ratings [$\text{DR}^{(k)}$]. The deterioration rates associated with this model are essentially the same. Therefore, the initial and terminal deterioration rates are sufficient to construct the corresponding project transition matrix.

$$\text{DR}(t) = -bt + c \tag{6}$$

where

$$P_{1,2} = P_{2,3} = P_{3,4} = \cdots = P_{m-1,m}$$

Decreasing Rate of Deterioration Model

The third type of prediction model is associated with progressively decreasing deterioration rates. The corresponding prediction model is a second-degree polynomial as defined by Equation 7. The second derivative for this model is positive, indicating that the corresponding parabolic performance curve is concave downward with a consistently decreasing slope. Similarly, the model coefficients a, b, and c are to be estimated from the predicted distress ratings [$\text{DR}^{(k)}$] with best-fitting or regression techniques.

$$\text{DR}(t) = at^2 - bt + c \tag{7}$$

The prediction model indicated by Equation 7 requires progressively decreasing deterioration rates. This performance trend typically occurs when the initial deterioration rate ($P_{1,2}$) is larger than the middle deterioration rate ($P_{x,x+1}$), which is, in turn, larger than the terminal deterioration rate ($P_{m-1,m}$). Equation 8a generates the deterioration rates ($P_{i,i+1}$) for condition states falling between (2) and ($x-1$) on the basis of the assumption that deterioration rates uniformly decrease from an initial value of ($P_{1,2}$) to a terminal value of ($P_{x,x+1}$). Similarly, Equation 8b yields the deterioration rates ($P_{i,i+1}$) for condition states falling between ($x + 1$) and ($m - 2$), assuming that the deterioration rates ($P_{i,i+1}$) uniformly decrease from an initial value of ($P_{x,x+1}$) to a terminal value of ($P_{m-1,m}$).

$$P_{i,i+1} = P_{1,2} - (i-1)\left(\frac{P_{1,2} - P_{x,x+1}}{x-1}\right) \qquad (i = 2, 3, \ldots, x-1) \tag{8a}$$

$$P_{i,i+1} = P_{x,x+1} - (i-x)\left(\frac{P_{x,x+1} - P_{m-1,m}}{m-x-1}\right) \qquad (i = x+1, x+2, \ldots, m-2) \tag{8b}$$

where

$P_{1,2} > P_{2,3} > \ldots > P_{x,x+1} > P_{x+1,x+2} > \ldots > P_{m-1,m}$

Zero Rate of Deterioration Model

The fourth type of prediction model is essentially a hypothetical one, wherein the pavement distress rating remains unchanged over time, indicating constant performance. The prediction model associated with this type of performance is indicated by Equation 9. The pavement distress rating remains equal to the maximum distress rating value that can be derived from Equation 3. The deterioration rates ($P_{i,i+1}$) associated with this prediction model are all equal to zero, and the transition probabilities ($P_{i,i}$) are all equal to 1. Therefore, the transition matrix associated with this model is simply the identity matrix.

$$\mathrm{DR}(t) = B_1 = \text{constant} \tag{9}$$

where

$P_{1,1} = P_{2,2} = P_{3,3} = \cdots = P_{m,m} = 1.0$ and

$P_{1,2} = P_{2,3} = P_{3,4} = \cdots = P_{m-1,m} = 0.0$

The deterioration rates ($P_{i,i+1}$) can be estimated using only the initial and terminal deterioration rates. In this case, the middle transition probability ($P_{x,x+1}$) is estimated as the average of the initial and terminal transition probabilities. Equations 5 and 8 are then used as outlined to estimate the remaining transition probabilities. Also, Equations 5 and 8 can be used to estimate the remaining transition probabilities for models other than polynomials, provided that these models are associated with either consistently increasing or decreasing deterioration rates.

Figure 2 shows sample pavement performance curves derived from only the initial and terminal transition probabilities in 10 condition states. The mean state distress ratings (B_i) are assumed to be 95, 85, . . . , 5 for condition states 1, 2, . . . , 10, respectively. The deterioration model for the concave-up curve is provided in Equation 10 with almost perfect R^2. The uniform deterioration model is presented in Equation 11 with perfect R^2. The deterioration model for the concave-down curve is indicated by Equation 12. The deployed length of time interval d is 1 year.

$$\mathrm{DR}(t) = -0.124t^2 - 1.758t + 94.938 \qquad (R^2 = 0.99) \tag{10}$$

$$\mathrm{DR}(t) = -3.984t + 94.929 \qquad (R^2 = 1.00) \tag{11}$$

$$\mathrm{DR}(t) = 0.200t^2 - 7.447t + 94.107 \qquad (R^2 = 0.99) \tag{12}$$

Stochastic-Based Approach for Predicting Flexible Pavement Remaining Strength

The proposed approach for predicting the remaining strength of flexible pavements is mainly dependent on the performance curves generated with the outlined stochastic approach. A distinct performance curve or its equivalent prediction model [DR(t)] can be developed for each pavement project considering a terminal service life of T years. The performance prediction models will mainly be used to estimate the future structural capacity associated with a particular pavement structure at a specified service time t.

Estimation of Initial and Future Structural Capacity

The initial structural capacity associated with a particular pavement structure can be defined with Equation 13. The structural capacity

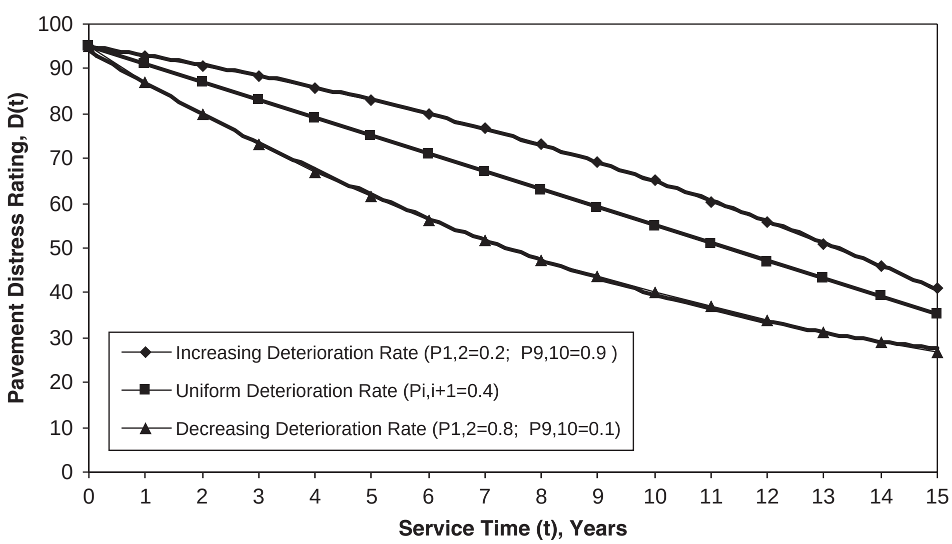

FIGURE 2 Sample pavement distress rating models predicted with stochastic modeling.

of the various pavement layers can be determined with appropriate relative strength indicators, such as the structural number and gravel equivalent (GE) deployed by AASHTO and the California Department of Transportation (Caltrans), respectively (*4, 15*).

$$\mathrm{SC}_p(0) = \mathrm{SC}_a(0) + \sum_j \mathrm{SC}_j(0) \qquad (13)$$

where

$\mathrm{SC}_p(0)$ = initial structural capacity of the pavement structure,
$\mathrm{SC}_a(0)$ = initial structural capacity of the asphalt layer, and
$\mathrm{SC}_j(0)$ = initial structural capacity of the *j*th underlying pavement layer.

The future structural capacity associated with a particular pavement structure at a given service time (t) is defined, as indicated by Equation 14, to be the product sum of multiplying the initial structural capacity of various pavement layers by their corresponding structural capacity adjustment factors. The structural capacity adjustment factor simply defines the remaining structural capacity for a particular pavement layer as a fraction of its initial structural capacity.

$$\mathrm{SC}_p(t) = \mathrm{SC}_a(0) \times \mathrm{SAF}_a(t) + \sum_j \mathrm{SC}_j(0) \times \mathrm{SAF}_j(t) \qquad (14)$$

where

$\mathrm{SC}_p(t)$ = future structural capacity of the pavement structure at service time (t),
$\mathrm{SAF}_a(t)$ = structural capacity adjustment factor for asphalt layer at service time (t), and
$\mathrm{SAF}_j(t)$ = structural capacity adjustment factor for the *j*th underlying pavement layer at service time (t).

The structural capacity adjustment factor associated with the asphalt concrete layer is considered to be the most critical factor in predicting the pavement's remaining structural capacity. The asphalt layer typically endures most of the strength degradation caused by traffic action, whereas the underlying granular layers rarely undergo significant strength degradation resulting from traffic action. However, adverse environmental and drainage conditions can contribute to the rapid deterioration of the entire pavement structure.

Estimation of Structural Capacity Adjustment Factors

The structural capacity adjustment factor associated with the asphalt layer can be estimated from the performance curve generated for a particular pavement structure. The transition probabilities used in generating the corresponding performance curve are typically estimated from a survey of pavement distress defects that mainly indicate strength degradation endured by the asphalt layer. The area falling under the performance curve has long been recognized as a direct measure of the pavement relative structural capacity (*4, 5, 13, 14*). Therefore, the area falling under the remaining service period (Δt) is directly used in Equation 15 to estimate the structural capacity adjustment factor associated with the asphalt layer. Figure 3 shows a typical performance curve, with the area corresponding to the remaining service period (Δt) shown with a shaded background.

$$\mathrm{SAF}_a(t) = \frac{\mathrm{AUC}_a(\Delta t)}{\mathrm{AUC}_a(T)} \qquad (15)$$

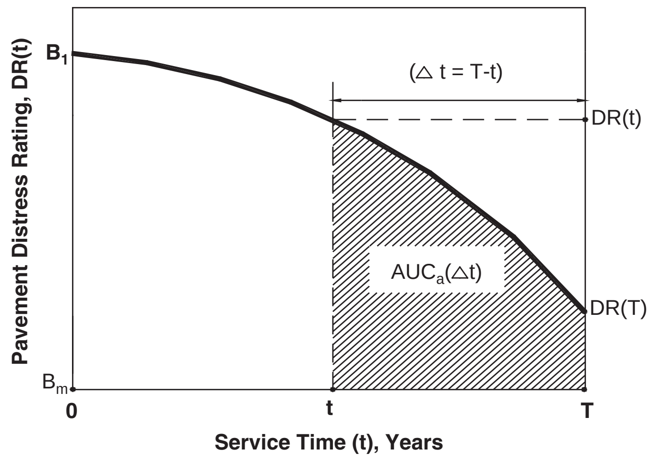

FIGURE 3 Proposed pavement remaining strength prediction model.

where

$0.0 \le (\Delta t = T - t) \le T$,
$\mathrm{SAF}_a(0) = 1.0$,
$\mathrm{SAF}_a(T) = 0.0$,
$\mathrm{AUC}_a(\Delta t)$ = area under performance curve corresponding to remaining service period (Δt),
Δt = remaining service period defined as the difference between terminal service life (T) and service time (t), and
$\mathrm{AUC}_a(T)$ = total area under performance curve corresponding to a terminal service life T.

The area under the generated performance curve can be estimated using either the predicted distress ratings [$\mathrm{DR}^{(k)}$] derived from Equation 3 or the equivalent distress prediction model [DR(t)] obtained from best-fitting or regression techniques. In the first option, the required area is obtained as a summation of the corresponding trapezoidal strip areas, wherein the ordinates of each trapezoidal strip are defined by two successive distress ratings, [$\mathrm{DR}^{(j)}$] and [$\mathrm{DR}^{(j+1)}$], with the width of the strip equal to the length of the time interval in years (d). In the second option, a direct integration of the distress prediction model is performed to obtain the required area. Equation 16 presents the mathematical formulations for the two outlined options that are required to obtain the area falling under the performance curve portion corresponding to the remaining service period (Δt).

$$\mathrm{AUC}_a(\Delta t) = \int_t^T \mathrm{DR}(t)\,dt = \frac{d}{2}\left(\mathrm{DR}^{(k)} + \mathrm{DR}^{(n)} + 2\sum_{j=k+1}^{n-1} \mathrm{DR}^{(j)}\right) \qquad (16)$$

where

$k = t/d$ = number of transitions corresponding to a service time t in years and
$n = T/d$ = number of transitions corresponding to a terminal service life T in years

Similarly, the total area falling under the entire performance curve for a terminal service life T can be estimated as indicated by Equation 17 using the two outlined integration and summation approaches. Once the two required performance areas are estimated through Equations 16 and 17, the structural capacity adjustment factor for the asphalt layer can be determined as defined in Equation 15.

$$\mathrm{AUC}_a(T) = \int_o^T \mathrm{DR}(t)\,dt = \frac{d}{2}\left(\mathrm{DR}^{(0)} + \mathrm{DR}^{(n)} + 2\sum_{j=1}^{n-1}\mathrm{DR}^{(j)}\right) \qquad (17)$$

The structural capacity adjustment factors associated with the underlying pavement layers can be estimated as indicated by Equation 18. Underlying granular layers would generally experience little or no strength degradation under normal traffic, environmental, and drainage conditions. However, adjustment factors can be estimated with layer relative strength coefficients similar to the AASHTO relative strength coefficients (a_j) or Caltrans GE factors (G_{f_j}). These layer coefficients have been correlated to typical support capacity indicators, such as the California bearing ratio or resilient modulus (M_R) obtained from testing underlying layer materials (*4*, *15*).

$$\mathrm{SAF}_j(t) = \frac{\mathrm{RSC}_j(t)}{\mathrm{RSC}_j(0)} \le 1.0 \qquad (18)$$

where

$0 \le t \le T$ = 1.0,
$\mathrm{SAF}_j(0)$ = 1.0,
$\mathrm{RSC}_j(t)$ = design relative strength coefficient for the *j*th underlying pavement layer at service time (t), and
$\mathrm{RSC}_j(0)$ = initial design relative strength coefficient for the *j*th underlying pavement layer.

Estimation of Pavement Remaining Strength

The pavement remaining strength can be estimated using a pavement structural capacity loss indicator [PSL(t)] defined by Equation 19. The pavement structural capacity loss, [PSL(t)], simply defines the percentage of strength lost over a specified service time t for a particular pavement structure. It is determined from the difference between the initial pavement structural capacity [$\mathrm{SC}_p(0)$] and the future pavement structural capacity [$\mathrm{SC}_p(t)$] estimated at a specified service time t. The estimated structural capacity loss can be used in developing guidelines for pavement M&R strategies and for establishing rehabilitation priority scheduling among pavement projects in the same roadway classification.

$$\mathrm{PSL}(t) = \frac{\mathrm{SC}_p(0) - \mathrm{SC}_p(t)}{\mathrm{SC}_p(0)} \times 100\% \qquad (19)$$

Estimation of Overlay Design Thickness

Another important application of the estimated future pavement structural capacity is obtaining a design thickness for asphalt resurfacing (overlay) required at a specified service time (t), as indicated by Equation 20. The estimated overlay design thickness is the thickness equivalent to the structural capacity loss the pavement structure has endured over a specified service time (t). Therefore, the required overlay structural capacity [$\mathrm{SC}_o(t)$] is set equal to the structural capacity loss determined as the difference between the initial structural capacity [$\mathrm{SC}_p(0)$] and the future structural capacity [$\mathrm{SC}_p(t)$]. The relative strength coefficient required by Equation 20 is similar to the AASHTO relative strength coefficient (a_1) or Caltrans GE factor (G_{f_1}) for asphalt materials (*4, 15*).

$$h_o(t) = \frac{\mathrm{SC}_o(t)}{\mathrm{RSC}_o} = \frac{\mathrm{SC}_p(0) - \mathrm{SC}_p(t)}{\mathrm{RSC}_o} \qquad (20)$$

where $h_o(t)$ is the required asphalt overlay thickness at service time (t) and RSC_o is the relative strength coefficient for asphalt overlay material.

SAMPLE APPLICATION

A sample application for predicting flexible pavement remaining strength is presented in this section. The sample application involves 12 village access roads selected from the northern districts of the West Bank of Palestine. These roads were constructed under the administration of the Palestinian Authority during the period of 1997–1998 with international donor funding provided to assist the Palestinian people in rebuilding their infrastructure system. The 12 roads to be investigated have been randomly selected from a larger sample of more than 50 roads that were built during the same period. These roads are mainly two-lane low-volume rural roads with lengths ranging from 3 to 7 km and are used by local residents to reach the nearest main highways. The sample application is to cover the three main objectives outlined earlier in the paper.

Sample Pavement Performance Prediction Models

A 10×10 transition matrix similar to the one outlined in Equation 2 has been used to represent the deterioration mechanism of each investigated roadway. Condition states are defined using equal 10 points distress rating (DR) range on an overall scale of 100 points, with higher ratings indicating better pavements. Pavement sections of 15-m lane length and 3-m width were used in the distress assessment. The DR assigned to a pavement section has been estimated using Equation 21 on the basis of the section surface area (A), cracked areas (A_c), deformed areas (A_d), cracking severity factor (F_c), and deformation severity factor (F_d) (*12*). Severity factors are assigned the values of 1, 2, and 3 for low, medium, and high levels of severity, respectively, on the basis of crack width and deformation depth. A defected area can only be counted as cracked or deformed. A sample value of (DR = 61.4) is obtained given $A = 45 - m^2$, $A_c = 14.2 - m^2$ ($F_c = 2$, medium severity), $A_d = 7.8 - m^2$ ($F_d = 1$, low severity), and $A_d = 5.3 - m^2$ ($F_d = 3$, high severity).

$$\mathrm{DR} = \left(\frac{3A - \sum F_c A_c - \sum F_d A_d}{3A}\right) \times 100 \qquad (21)$$

where

$\Sigma A_c + \Sigma A_d$ is less than or equal to A.

The initial ($P_{1,2}$) and terminal ($P_{9,10}$) transition probabilities have been estimated for each investigated roadway by conducting two cycles of pavement distress assessment. The two cycles were separated by a 1-year time interval representing the length of the deployed transition. Equation 22 has been used to estimate the transition probabilities on the basis of the number of pavement sections found in state (i) after the first cycle ($N_i^{(1)}$) and second cycle ($N_i^{(2)}$), (*11, 12*). A minimum of 30 pavement sections were surveyed in the first cycle for each condition state (States 1 and 9) and were inspected again in

TABLE 1 Sample Pavement Deterioration Rates and Original Design Parameters

Road No.	$P_{1,2}$	$P_{9,10}$	h_a (cm)	h_1 (cm)	R-Value	$W_{80}\times 10^3$	Drainage Condition
1	0.18	0.38	8	25	46	470	Good
2	0.25	0.36	8	25	41	320	Good
3	0.26	0.69	8	30	30	350	Fair
4	0.29	0.51	7	30	27	180	Fair
5	0.32	0.74	7	35	24	250	Fair
6	0.35	0.31	8	30	40	520	Good
7	0.38	0.49	8	35	28	650	Fair
8	0.39	0.75	8	40	21	700	Fair
9	0.47	0.85	9	45	10	620	Poor
10	0.52	0.49	8	35	26	580	Fair
11	0.58	0.52	9	40	16	660	Poor
12	0.64	0.50	9	45	13	750	Poor

the second cycle. The sections are assigned to the states according to their estimated distress ratings. Sample values of ($P_{1,2} = 0.18$) are obtained given ($N_1^{(1)} = 50$, $N_1^{(2)} = 41$), and ($P_{9,10} = 0.38$) given ($N_9^{(1)} = 32$, $N_9^{(2)} = 20$).

$$P_{i,i+1} = \frac{N_i^{(1)} - N_i^{(2)}}{N_i^{(1)}} \tag{22}$$

The estimated initial ($P_{1,2}$) and terminal ($P_{9,10}$) transition probabilities are provided in Table 1 for the 12 roadways under investigation. Examination of the tabulated transition probabilities reveals that the increasing deterioration rate model as represented by Equation 5 is valid when the initial probability ($P_{1,2}$) is smaller than the terminal probability ($P_{9,10}$). Similarly, the decreasing deterioration rate model as defined by Equation 8 is applicable when the initial probability ($P_{1,2}$) is larger than the terminal probability ($P_{9,10}$). Then, the performance curves (models) are developed using the predicted distress ratings [$DR^{(k)}$] generated for each roadway, as indicated by Equation 3. A terminal service life T of 20 years has been used, which is equivalent to 20 transitions (n), since one discrete-time interval (d) is equal to 1 year. The predicted pavement distress ratings are to be used in estimating the structural capacity adjustment factor associated with the asphalt layer to predict the flexible pavement remaining strength. Table 1 provides additional data pertaining to the original roadway design, which include the asphalt layer thickness (h_a), granular base layer thickness (h_1), subgrade resistance value (R), design 80kN (18K) equivalent single axle load applications (W_{80}), and current roadway drainage condition rating. The qualitative roadway drainage condition rating is assigned according to the condition of the existing roadway drainage system, quality of roadway transverse and longitudinal profiles, and type of surrounding terrain (level, rolling, or mountainous).

Predicting Remaining Strength of Sample Flexible Pavement

The predicted pavement distress ratings [DR$^{(k)}$] have been used to estimate the principal structural capacity adjustment factor [SAF$_a(t)$] associated with the asphalt layer as defined by Equation 15, using a service time (t) of 10 years. The two areas under the performance curve [AUC$_a$(10)] and [AUC$_a$(20)] required by Equation 15 have been also estimated, as indicated by Equations 16 and 17, respectively, using the generated distress ratings [DR$^{(k)}$]. Table 2 provides the values for the two areas under the performance curve and the corresponding asphalt structural capacity adjustment factor. The sample results provided indicate that the values associated with the asphalt structural capacity adjustment factor range from 0.180 to 0.422. This basically means that the remaining strength for the asphalt layer after 10 years of service ranges from 18.0% to 42.2% for the investigated roadways, with an average value of about 30%. This might be somewhat lower than expected, which could be attributed to inadequacy of construction practices and of quality assurance policies that is typically encountered in developing countries. The structural capacity adjustment factor for the granular base layer [SAF$_1$(10)] is assumed as provided in Table 2, based on the qualitative roadway drainage condition rating of 1.0 for good, 0.9 for fair, and 0.8 for poor. Alternatively, the base strength adjustment factor can be estimated as outlined in Equation 18 using original and present layer relative strength coefficients.

TABLE 2 Sample Pavement Structural Capacity Adjustment Factors Predicted with 10 Years' Service Time

Road No.	$P_{1,2}$	$P_{9,10}$	AUC$_a$ (10)	AUC$_a$ (20)	SAF$_a$ (10)	SAF$_1$ (10)
1	0.18	0.38	621.25	1,473.35	0.422	1.0
2	0.25	0.36	537.55	1,358.10	0.396	1.0
3	0.26	0.69	395.25	1,191.80	0.332	0.9
4	0.29	0.51	430.65	1,224.20	0.352	0.9
5	0.32	0.74	304.00	1,066.70	0.285	0.9
6	0.35	0.31	445.90	1,223.30	0.364	1.0
7	0.38	0.49	333.60	1,083.40	0.308	0.9
8	0.39	0.75	230.45	956.55	0.241	0.9
9	0.47	0.85	149.50	828.70	0.180	0.8
10	0.52	0.49	229.00	919.55	0.249	0.9
11	0.58	0.52	188.55	855.45	0.220	0.8
12	0.64	0.50	167.00	811.95	0.206	0.8

The initial structural capacity for the two-layer pavement structures is estimated using the relative strength indicator known as GE, deployed by Caltrans in the design of flexible pavement structures (*15*). The initial structural capacities associated with asphalt concrete and aggregate base layers, [$SC_a(0)$] and [$SC_1(0)$], are assumed to be equal to their corresponding GE values, which are obtained from multiplying the layer thickness in feet by the layer GE factor, with results provided in Table 3. The GE factors for asphalt concrete and aggregate base (G_{f_a} and G_{f_1}) are assumed to be equal to 2.0 and 1.2, respectively. The GE factor for asphalt concrete is dependent on the traffic index, which is a function of the 80k*N* (18*K*) equivalent single axle load applications. Table 3 provides the initial pavement structural capacity [$SC_p(0)$], as proposed by Equation 13, and the predicted pavement structural capacity [$SC_p(10)$], as proposed by Equation 14, at 10 years of service. Both the initial and predicted pavement structural capacities are used to determine the overall pavement structural capacity loss [PSL(10)], as presented in Equation 19, and required overlay design thickness [$h_o(10)$], as presented by Equation 10, at 10 years of service. The tabulated results show that the [PSL(10)] has ranged from 19.58% to 35.83% with an average value of 27.16%, and the overlay design thickness has ranged from 4.6 to 12.8 cm with an average value of about 8 cm. The asphalt concrete relative strength coefficient (RSC_o) used in the calculation of the overlay design thickness is the GE factor with a 2.0 value.

Pavement rehabilitation strategies have been recommended in line with the [PSL(10)] and [$h_o(10)$] values, as indicated in Table 3, which include plain overlay (PO), skin patch (SP), and reconstruction (RE). PO is to be applied directly to the existing surface after crack sealing and RE of localized failures, with estimated overlay design thickness rounded to the nearest 1 cm. SP involves milling about half of the existing asphalt layer, crack sealing and RE of localized failures, and placement of a new asphalt layer with thickness equal to the estimated overlay design thickness reduced by 1 cm for every 2 cm of the remaining original asphalt layer. RE includes removal of the existing asphalt layer and placement of a 10-cm leveling aggregate layer and a new asphalt layer with thickness equal to the estimated overlay design thickness reduced by 1 cm for every 2 cm of the leveling aggregate layer, provided that the final thickness is not lower than the original asphalt layer thickness. The pavement structural capacity loss [PSL(10)] can also be used to assign priority listings among rehabilitation project candidates in the same roadway classification, as provided in Table 3.

Sample Empirical Stochastic-Based Prediction Models

The main outcome of the demonstrated stochastic modeling for predicting flexible pavement remaining strength is the principal structural capacity adjustment factor [$SAF_a(t)$] associated with the asphalt layer. Relevant empirical models generated using multiple linear regression techniques can be very useful to practitioners. Such models can be used to estimate the [$SAF_a(t)$] factor from related variables—such as design load applications (W_{80}), subgrade resistance value (R), total pavement thickness (h_p), and roadway drainage (D)—which are assigned condition ratings of 1 for good, 2 for fair, and 3 for poor. A sample regression model with nonlinear transformations for estimating the [$SAF_a(10)$] factor is presented in Equation 23, with relevant input data given in Table 1 and the corresponding [$SAF_a(10)$] values in Table 2. The generated model is significant at more than a 99% confidence level with the variable coefficients significant at 98%. The model has a 94.35% determination coefficient (R^2) and 0.022 standard error of estimate.

$$SAF_a(10) = 0.858 - 0.346 \log W_{80}^{0.5} + 1.31 \times 10^{-6} \left(\frac{R}{D}\right)^3 - 0.194 \log \left(\frac{R}{h_p}\right)^2 \qquad (23)$$

Another equally important empirical model that can be of interest to practitioners is one that estimates the overlay design thickness ($h_o(10)$) directly from related variables. Equation 24 presents a sample overlay model derived from the estimated overlay design thicknesses in Table 3 and the corresponding input data in Table 1. The developed regression model is significant at more than 99% confidence level with the variable coefficients significant at 99.9%. The model has a 99.56% determination coefficient and 0.213 standard

TABLE 3 Sample Pavement Initial and Remaining Strength Indicators and Overlay Design Thickness with 10 Years' Service Time

Road No.	$SC_a(0)$	$SC_1(0)$	$SC_p(0)$	$SC_p(10)$	PSL(10)	$h_o(10)$ (cm)	Repair Strategy	Priority Listing
1	0.525	0.984	1.509	1.206	20.08	4.6	PO	11
2	0.525	0.984	1.509	1.192	21.01	4.8	PO	10
3	0.525	1.181	1.706	1.237	27.49	7.2	SP	5
4	0.459	1.181	1.640	1.224	25.36	6.3	PO	9
5	0.459	1.378	1.837	1.371	25.37	7.1	SP	8
6	0.525	1.181	1.706	1.372	19.58	5.1	PO	12
7	0.525	1.378	1.903	1.402	26.33	7.6	SP	7
8	0.525	1.575	2.100	1.544	26.48	8.5	SP	6
9	0.591	1.772	2.363	1.524	35.51	12.8	RE	2
10	0.525	1.378	1.903	1.371	27.96	8.1	SP	4
11	0.591	1.575	2.166	1.390	35.83	11.8	RE	1
12	0.591	1.772	2.363	1.539	34.87	12.6	RE	3

error of estimate. Similar empirical models have been attempted to estimate the initial and transition probabilities from the same related variables but resulted in much lower confidence levels and determination coefficients.

$$h_o(10) = 3.742 + 0.060(h_p D) - 3.215\left(\frac{RD}{W_{80}}\right)^{0.5} \quad (24)$$

The presented sample empirical stochastic-based models can help practicing pavement engineers save time and money, provided that the models are applied to roadway conditions similar to those used in the models' development.

CONCLUSIONS AND RECOMMENDATIONS

The presented sample results have indicated that stochastic modeling can be useful in predicting flexible pavement remaining strength. The simplified approach outlined for predicting pavement distress ratings using only the initial and terminal transition probabilities is certainly an advantage to local highway agencies with limited resources. Estimates of these two transition probabilities can be obtained by referring to historical pavement distress records or conducting two cycles of pavement distress assessment. Once such estimates are available, the approach presented for predicting flexible pavement remaining strength is a straightforward procedure with minimal data requirements. It is recommended that 10 condition states be used in forming a project transition matrix. Prediction of remaining strength is to be performed with an appropriate flexible pavement design procedure, such as the demonstrated Caltrans procedure, which deploys the GE as an indicator of the pavement relative structural capacity.

The predicted flexible pavement structural capacity can be easily used in estimating the required overlay design thickness, recommending appropriate rehabilitation strategies, and establishing rehabilitation project priority listing. Implementation of the presented stochastic model, which forms the basis for generating the required project performance curves, is mainly dependent on simple field surveys and measurements of pavement distress defects. The cost associated with conducting the required distress assessment is very minimal compared with the savings that can be expected once adequate records become available to develop relevant empirical remaining strength models similar to the ones presented in the sample application. The derived empirical stochastic-based models can provide highway agencies with substantial savings, provided that they are applied to roadway conditions similar to those used in developing the models. A highway agency may develop different sets of empirical models for various roadway classifications of a particular roadway network, which permits the development of pavement rehabilitation and management policies at the network level.

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