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Deterministic Performance Prediction Model for Rehabilitation and Management of Flexible Pavement

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A deterministic performance prediction model for use in rehabilitation and management of flexible pavements is presented. The model utilizes the serviceability concept adopted by the American Association of State Highway and Transportation Officials (AASHTO) for use in the design of flexible pavements. The serviceability concept is defined through the introduction of the present serviceability index (PSI) that is related to the cumulative axle load applications by means of a performance curve. The performance curve for a particular pavement structural section can be estimated using an incremental solution of the AASHTO basic design equation. The constructed performance curve can be used in several applications related to pavement rehabilitation and management. These applications include evaluating potential pavement design and rehabilitation alternatives using appropriate performance indicators, developing asphaltic overlay design procedure, performing life-cycle analysis and pavement management applications. Pavement management applications include prediction of future pavement conditions and estimation of transition probabilities used in stochastic prediction models. The presented analyses and techniques are potentially useful for pavement engineers and can be used as an effective teaching tool for pavement design students.

Keywords: Pavement performance prediction; Pavement performance trends; Pavement design and rehabilitation; Asphaltic overlay design; Pavement life-cycle analysis; Pavement management

INTRODUCTION

Application of effective techniques in modeling flexible pavement rehabilitation and management requires pavement performance condition feedback. The needed performance feedback is typically obtained from field measurements of pavement distress conducted using appropriate inspection procedures. The field measurements are usually performed annually or biennially on pavement systems with similar material properties and loading conditions. The collected pavement distress data is then used to study the performance of pavements over time, and to predict the future performance of similar pavements. Prediction of future pavement condition is an essential factor for effective application of any pavement rehabilitation and management model.

Pavement performance prediction models are grouped into two classes: probabilistic and deterministic (Robinson *et al.*, 1998). The probabilistic model is a probability function that predicts the future pavement condition with a certain level of probability. Probability levels are assigned to possible future condition outcomes based on engineering judgment or from an analysis of past performance of pavements (Butt *et al.*, 1987; Kerali and Snaith, 1992; Shahin, 1994). The deterministic model is a mathematical function that predicts the future pavement condition as a precise value. The associated mathematical function is derived from observed or measured pavement deterioration using mechanistic, regression, or mechanistic–empirical methods. Several researchers have developed a wide variety of deterministic prediction models based on the three listed methods (Shahin *et al.*, 1987; Watanatada *et al.*, 1987; George *et al.*, 1989; Gopinath *et al.*, 1994).

The presented deterministic performance prediction model has been developed using an incremental analysis of the American Association of State Highway and Transportation Officials (AASHTO) basic design equation used in the design of flexible pavements. The AASTHO basic design equation is empirically derived from the AASHO Road Test using regression techniques (AASHTO, 1993). The outcome of the developed prediction model is the generation of a unique performance curve for a given pavement structure.

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The generated performance curve provides a simple tool to estimate the pavement performance condition at any given future time. It can especially be used in the absence of actual pavement performance condition data. The two main parameters defining a pavement performance curve are the present serviceability index (PSI) and 80 kN equivalent single axle load (ESAL) applications. These two parameters are also related to materials properties, drainage and environmental conditions, reliability and prediction variations, and performance trends. The design approach applies all related parameters to obtain a measure of the required structural strength through an index known as the structural number (SN). Eq. (1) provides the basic equation used for the design of flexible pavement (AASHTO, 1993)

$$\log W_T = Z_R S_o + 9.36 \log(SN+1) + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log(M_R) - 8.27 \quad (1)$$

where:

W_T = number of 80 kN ESAL applications estimated for a selected design period and design lane,

Z_R = standard normal deviate for a specified reliability level,

S_o = combined standard error of the traffic prediction and performance prediction,

ΔPSI = difference between the initial PSI (P_o) and the terminal serviceability index (P_t),

SN = design structural number indicative of the total required pavement thickness, and

M_R = subgrade resilient modulus in pound per squared inch.

Once the design SN is determined from Eq. (1), it is then converted to layer thickness using Eq. (2)

$$\mathrm{SN} = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 \quad (2)$$

where a_1, a_2 and a_3 are the layer relative strength coefficients, m_2 and m_3 are the layer drainage coefficients, and D_1, D_2 and D_3 are the layer thickness for surface, base and sub-base, respectively. Eq. (2) provides large number of feasible solutions in terms of layer thickness. However, any selected solution must satisfy minimum thickness for surface and base layers recommended by AASHTO.

METHODOLOGY

In this section, a detailed description of the mathematical procedure developed for use in the deterministic performance prediction model is presented based on the AASHTO serviceability concept, followed by a sample application.

Performance Prediction Model

In the design mode and after all related parameters are estimated, Eq. (1) is solved for the design SN by trial and error or using the design chart found in AASHTO (1993). The approach used to define a pavement performance curve as a function of the PSI and 80 kN ESAL applications or service time is based on the direct use of Eq. (1). The incremental 80 kN ESAL applications (W_i) is calculated by specifying varying values of the incremental change in the PSI (ΔPSI_i). The incremental change in the PSI is defined as the difference between the initial serviceability index (P_o) and the incremental PSI (PSI_i). The incremental PSI is varied between its assigned initial value and its terminal one.

Figure 1 shows the basic concept wherein the incremental change in PSI (ΔPSI_i) is plotted against the corresponding incremental load applications (W_i). The estimated incremental load applications need to be converted into an equivalent incremental service time (T_i).

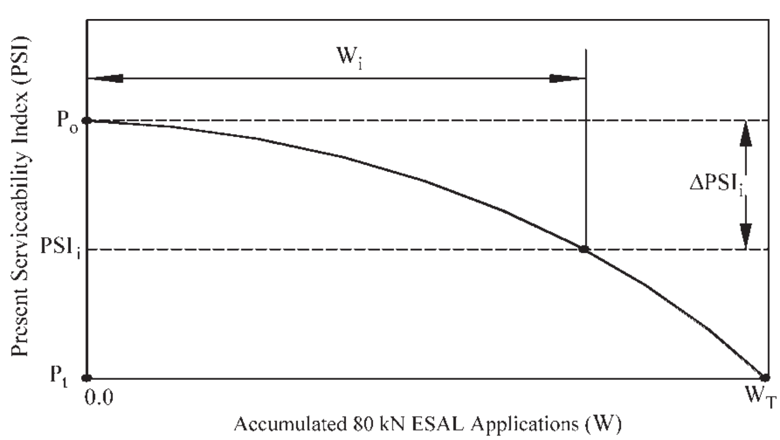

FIGURE 1 Typical pavement performance curve.

A logarithmic relation can be established between service time and load applications. The design load applications (W_T) is typically estimated based on the initial year 80 kN ESAL applications (W_o) as indicated in Eq. (3a) with T being the design service life and $G(T)$ is the traffic growth factor for T years. The growth factor at any given future time can be estimated using the popular formula recommended by the Asphalt Institute (AI, 1991) as provided in Eq. (3b).

Now, estimating the incremental load applications (W_i) at any incremental service time (T_i) can be obtained as presented in Eq. (3c) using the incremental growth factor $G(T_i)$. The initial year 80 kN ESAL applications (W_o) and the incremental growth factor $G(T_i)$ as derived from Eqs. (3a) and (3b), respectively, are substituted in Eq. (3c) to yield Eq. (3d). Equation (3d) is then solved for the incremental service time (T_i) which requires taking the logarithm of both sides of the equation to yield Eq. (3e). Therefore, Eq. (3e) can be used to estimate the incremental service time for any given incremental load applications using only the design service period (T), design load applications (W_T), and annual traffic growth rate (r) in decimal form. Equation (3e) provides a non-linear relation between service time and load applications that can effectively account for the actual increase in traffic loads, thus, generating a more accurate performance curve

$$W_T = W_o\, T\, G(T) \tag{3a}$$

$$G(T) = \frac{(1+r)^T - 1}{r\,T} \tag{3b}$$

$$W_i = W_o\, T_i\, G(T_i) \tag{3c}$$

$$W_i = \left(\frac{W_T}{T\,G(T)}\right)\left(\frac{(1+r)^{T_i} - 1}{r}\right) \tag{3d}$$

$$T_i = \frac{\mathrm{Log}\left(1 + \frac{W_i}{W_T}\left[(1+r)^T - 1\right]\right)}{\mathrm{Log}\,(1+r)}. \tag{3e}$$

The procedure to construct a performance curve for a particular pavement structure is outlined below:

(1) The SN for a new pavement design is estimated from Eq. (1) using the subgrade resilient modulus, the initial and terminal PSI values (P_o and P_t), the normal standard deviate (Z_R) and the combined standard error (S_o). The SN for an existing pavement structure can be obtained based on non-destructive testing (NDT) of pavement to yield estimates of the pavement layer moduli. The pavement layer relative strength coefficients can then be estimated from correlation charts and used in Eq. (2) to derive an estimate of the corresponding pavement SN.

(2) A selected number of data points is to be used in the construction of the performance curve. The difference between the initial and terminal PSI values is divided into a number of equal intervals (n) each of magnitude equal to a specified PSI change (ΔP). The number of data points generated will be equal to the number of intervals plus one. Equation (4) can be used to obtain the incremental PSI (PSI_i) value for all applicable data points

$$\mathrm{PSI}_i = P_o - (i-1)\Delta P, \quad i = 1, 2, \ldots, n+1. \tag{4}$$

(3) The incremental change in PSI is then determined using Eq. (5). The initial serviceability index (P_o) is fixed in Eq. (5) while the incremental terminal value (i.e. PSI_i) is varied as required by the AASHTO basic design equation

$$\Delta\mathrm{PSI}_i = P_o - \mathrm{PSI}_i \quad i = 1, 2, \ldots, n+1. \tag{5}$$

(4) The incremental load applications (W_i) is determined for each incremental PSI change ($\Delta\mathrm{PSI}_i$) using Eq. (1). The other required design input parameters are assumed to be the same as in step 1 for new pavement design. For existing pavement structures, the subgrade modulus value shall be the *in-situ* value obtained from NDT testing or any other appropriate testing procedure. Of course, the SN associated with the original pavement structure can be used if known.

(5) The incremental service time (T_i) is lastly determined from Eq. (3e) for each corresponding incremental load applications (W_i). A performance curve can now be constructed that relates the incremental service time to the incremental PSI.

Sample Application

Table I provides sample calculations for the various performance parameters involved in the above mathematical model. The first step is to determine the design SN based on the projected 80 kN ESAL applications as represented by W_T. A computer program has been designed to perform a sequential trial and error solution of Eq. (1) until it reaches very close results as specified by tolerance limits. Then, the procedure outlined earlier is used to determine the required performance curve parameters with results summarized in Table I. For this sample application, the specified PSI change (ΔP) is taken to be 0.5, the initial and terminal serviceability indices

TABLE I Estimation of pavement performance parameters for curve A

Point i	PSI_i	$\Delta\mathrm{PSI}_i$	W_i (10^6)	T_i (years)
1	4.5	0.0	0.000	0.00
2	4.0	0.5	0.423	9.44
3	3.5	1.0	0.611	13.13
4	3.0	1.5	0.759	15.85
5	2.5	2.0	0.884	18.05
6	2.0	2.5	1.000	20.00

Assumed design parameters are: $W_T = 1.0 \times 10^6$, $M_R = 105$ MPa (15,000 psi), SN = 2.57.

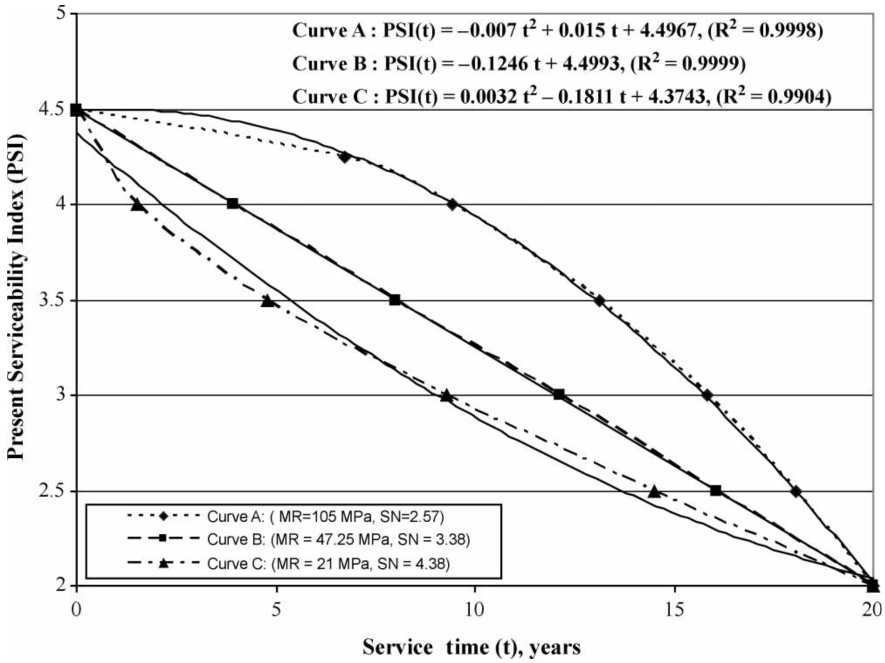

FIGURE 2 Sample performance curves estimated from AASHTO design method.

(P_o and P_t) specified as 4.5 and 2.0, respectively, the standard normal deviate (Z_R) is -1.645 for 95% reliability, the combined standard error (S_o) is 0.35, the annual traffic growth rate is 2%, and the design service life (T) is 20 years. These same values have been used in all subsequent sample applications.

The estimated performance curve for this example is obtained by plotting the incremental PSI (PSI_i) versus the incremental service time (T_i). The resulting performance curve is shown in Figure 2 as represented by Curve A using 105 MPa (15,000 psi) subgrade resilient modulus value and one million 80 kN ESAL applications. There are two additional Curves B and C shown in the same figure, which are obtained for the same traffic loading level but using smaller design resilient modulus values. Performance parameters for these two curves are generated in the same way and summarized in Table II along with their corresponding incremental deterioration rates and relative performance values to be defined in the next section.

PAVEMENT REHABILITATION AND MANAGEMENT

Pavement performance trends serve as an invaluable tool in helping the pavement engineer to make effective decisions in relation to pavement design and rehabilitation including pavement management. Performance curves reveal very crucial information that can directly be used in several related pavement applications. These applications include evaluating potential design and rehabilitation alternatives, estimating required asphaltic overlay thickness, performing life-cycle analysis and pavement management applications. All these potential applications are discussed in detail in the subsequent subsections.

Pavement Design and Rehabilitation Alternatives Evaluation

The AASHTO pavement design method applies several parameters as presented earlier. The impact of these

TABLE II Sample pavement deterioration rate and relative performance

		Curve A			Curve B			Curve C		
		SN = 2.57, M_R = 105 Mpa			SN = 3.38, M_R = 47.25 MPa			SN = 4.38, M_R = 21 MPa		
Point i	PSI_i	$\Delta T_{i,i+1}$ (years)	$R_{i,i+1}$	$A_{i,i+1}$	$\Delta T_{i,i+1}$ (years)	$R_{i,i+1}$	$A_{i,i+1}$	$\Delta T_{i,i+1}$ (years)	$R_{i,i+1}$	$A_{i,i+1}$
1	4.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	4.0	9.44	0.053	21.24	3.95	0.126	8.89	1.50	0.333	3.38
3	3.5	3.69	0.136	6.46	4.07	0.123	7.12	3.30	0.152	5.78
4	3.0	2.72	0.184	3.40	4.11	0.122	5.14	4.48	0.112	5.60
5	2.5	2.20	0.227	1.65	3.94	0.127	2.96	5.21	0.096	3.91
6	2.0	1.95	0.256	0.49	3.93	0.127	0.98	5.51	0.091	1.38
		$RP_A = \frac{33.24}{50.00} = 0.665$			$RP_B = \frac{25.09}{50.00} = 0.502$			$RP_C = \frac{20.05}{50.00} = 0.401$		

parameters on the overall pavement performance can be evaluated using the newly introduced performance indicators called deterioration rate (R) and relative performance (RP). The incremental deterioration rate ($R_{i,i+1}$) is defined as the slope of the line connecting points i and $i+1$ on the performance curve. Mathematically, it is stated as provided in Eq. (6).

$$R_{i,i+1} = \frac{\Delta \mathrm{PSI}_{i,i+1}}{\Delta T_{i,i+1}} = \frac{\mathrm{PSI}_{i+1} - \mathrm{PSI}_i}{T_{i+1} - T_i} \quad (6)$$

The presented incremental deterioration rate can be used in evaluating potential pavement design alternatives. It defines the shape of the performance curve in relation to its concavity. A performance curve is concave downwards if it is associated with successively increasing incremental deterioration rate values, an indication of superior pavement structure. Similarly, a performance curve is concave upwards if it is associated with successively decreasing incremental deterioration rate values, an indication of inferior pavement structure. A uniform incremental deterioration rate is an indication of a linear performance line.

Evaluation of pavement design alternatives can be made effectively by using a proposed performance indicator called relative performance. Performance is defined as the integral of the performance curve constructed using the PSI versus service time. Therefore, the area falling under the curve is, by definition, an indication of performance (Yoder and Witczak, 1975; Haung, 1993). Relative performance is defined as the ratio of the area corresponding to a particular curve to that of a perfect performance curve. A perfect performance curve is the one represented by a hypothetical horizontal straight line indicating constant PSI value over time. The maximum theoretical value of relative performance is unity. Figure 3 shows typical performance trends with their corresponding relative performance values (Abaza and Abu-Eisheh, 2003). The pavement design that provides the best performance is the one associated with the highest relative performance value. A 0.5 value is an indication of uniform incremental rate of deterioration, whereas a value below that is an indication of decreasingly lower rates of deterioration. Relative performance is mathematically stated in Eq. (7).

$$\mathrm{RP} = \frac{\sum_{i=1}^{n} A_{i,i+1}}{(P_o - P_t)T} \quad (7)$$

and,

$$A_{i,i+1} = \left[\frac{1}{2}(\mathrm{PSI}_i + \mathrm{PSI}_{i+1}) - P_t\right]\Delta T_{i,i+1}$$

where $A_{i,i+1}$ represents a trapezoidal strip area bounded by two curve points, other variables as defined earlier.

Table II provides sample calculations for both indicators. Three pavement designs have been used in the sample presentation provided in Table II. The three designs differ only in the assigned value of subgrade resilient modulus and consequently, the design SN. The design represented by Curve A shown in Figure 2 is associated with increasing incremental deterioration rates. This trend in deterioration is a favorable one in comparison to the trends demonstrated by Curves B and C. Curve B is associated with approximately uniform incremental rates of deterioration, an indication of linear performance trend. The trend of Curve C is opposite to that of Curve A with decreasing deterioration rates. The pavement engineer should avoid selecting a pavement design that exhibits a performance trend similar to that of Curve C, which can be done by improving the subgrade strength.

Performance of Curve B is obviously superior to that of Curve C but inferior to Curve A. The three curves are derived for the same level of loading condition, one million 80 kN ESAL applications, but are associated with different design SNs. The superior performance of Curve A is associated with the lowest design SN and highest subgrade resilient modulus, whereas the undesirable performance of Curve C has the highest design SN and lowest modulus values. This indicates that performance can be improved by enhancing the subgrade strength assuming that all other design input parameters remain unchanged. This conclusion emphasizes the significance of subgrade strength in improving the performance of flexible pavements. Best-fit equations have been derived for the three presented performance curves with almost perfect coefficient of determination (R^2) as shown in Figure 2.

The different trends associated with the three presented performance curves can be explained using the mechanistic approach for analyzing layered pavement systems. The mechanistic design parameter that is typically used is the vertical subgrade strain (or surface deflection), which is the main indicator for subgrade bearing strength. A two-layered pavement system consisting of a full depth asphalt concrete pavement has been analyzed using 40.05 kN (9 kips) single load with 490 kPa (70 psi) tire pressure.

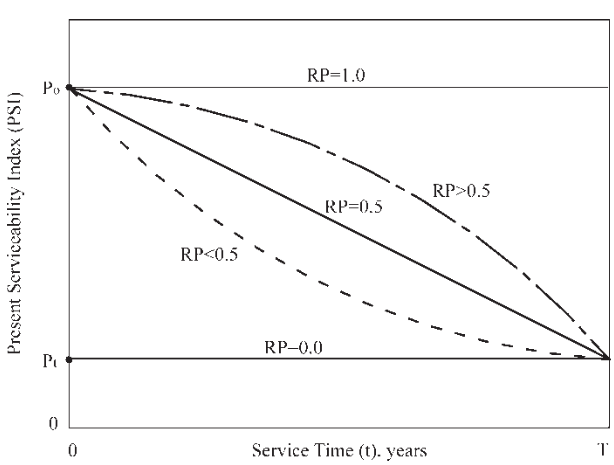

FIGURE 3 Performance curves with typical relative performance values.

The modulus value for the asphaltic pavement is assumed to be 1750 MPa (250,000 psi). The estimated maximum surface deflections associated with performance curves A, B and C, are 0.012, 0.023 and 0.043 cm, respectively. Therefore, the deflection results are in support of the performance trends depicted for the three performance curves presented above.

The relative performance has been calculated for the three Curves A, B and C shown in Figure 2 with calculation results summarized in Table II. Curve A has the highest relative performance amongst the three curves, an indication of superior performance. Curve B is associated with RP value of about 0.5, which indicates a uniform rate of deterioration, whereas Curve C has a value below 0.5, an indication of undesirable performance. Relative performance can be effectively used to check for unfavorable performance (i.e. $RP < 0.5$) and to select the best pavement design as indicated by the highest RP value. In addition, relative performance can be effective especially in evaluating potential pavement rehabilitation alternatives as presented later.

Similarly, it can be concluded that relative performance is directly related to the subgrade resilient modulus value for a particular flexible pavement structure when all other design input parameters are unchanged. This conclusion again indicates the significance of subgrade strength in the design of flexible pavements as would be expected. The sample results also indicate that relative performance is inversely related to the design SN.

Asphaltic Overlay Thickness Design

The estimation of the required asphaltic overlay thickness for rehabilitation of flexible pavement has been mostly based on experience (Yoder and Witczak, 1975; Haung, 1993). The majority of highway agencies apply deflection testing to determine the needed asphaltic overlay thickness as obtained from the backward solution of the multi-layered elastic theory. Occasionally, the derived solutions are inconclusive as to provide practical design thickness and then the engineer has to rely on experience. Deflection tests and related studies are typically associated with a high price tack as most of local agencies rely on private consultants to provide them with these services. Several local agencies have developed typical overlay thickness schedules using a prescription method.

A simplified procedure to estimate overlay thickness based on a generated pavement performance curve is proposed in this paper. The procedure attempts to compensate the existing pavement structural section for the loss in strength it has endured over a selected period of time. The loss in strength is assumed to be directly proportional to the utilized portion of the area under the performance curve. The required overlay SN (SN_{i+1}) is assumed to be directly proportional to ratio of the utilized curve area portion (A_{i+1}) over a specified service time (T_{i+1}) to the total area (A_T) falling under the performance curve. Eq. (8a) states that the loss in strength as determined from the performance curve is assumed to mainly affect the asphaltic layer strength (SN_1). However, the loss in strength resulting from the other pavement layers is accounted for using a reduction strength factor (S_{i+1}) estimated, based on experience and engineering judgment. For granular pavement layers, the strength reduction factor is typically small in the range 0.0–0.3, and it depends on service time and local field conditions.

Eq. (8b) applies a traffic growth adjustment factor (G_{i+1}) to the overlay thickness based on the assumption that overlay thickness is linearly proportional to the logarithmic change in load applications over service time (T_{i+1}). The number of load applications at the corresponding service time ($T + T_{i+1}$) is determined using Eq. (3). It must be emphasized that the estimated overlay thickness is intended to restore the pavement structure to its original condition considering a design service life of (T) years. Figure 4 shows the performance curve associated with the presented overlay model.

$$SN_{i+1} = \left(\frac{A_{i+1}}{A_T} SN_1\right) + (SN - SN_1)\, S_{i+1} \qquad (8a)$$

$$D_{i+1} = \left(\frac{SN_{i+1}}{a_1}\right) G_{i+1} = \left(\frac{SN_{i+1}}{a_1}\right)(1 + \log W_{T+T_{i+1}} - \log W_T) \qquad (8b)$$

where:

$$A_{i+1} = \sum_i A_{i,i+1}, \quad A_T = \sum_{i=1}^{n} A_{i,i+1}, \quad 0 \le T_{i+1} \le T$$

where:

SN_{i+1} = overlay SN required at T_{i+1} service time,

SN_1 = structural number of the original asphaltic layer,

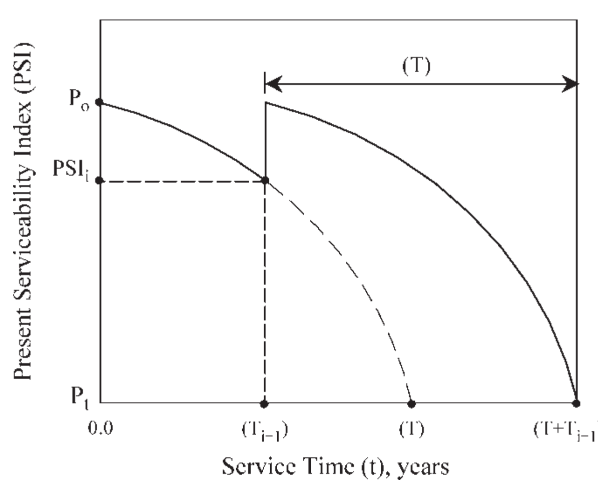

FIGURE 4 Performance curve associated with overlaid pavement.

TABLE III Asphaltic overlay thickness sample presentation

	Example 1						Example 2					
	$W_T = 1.0 \times 10^6$, $M_R = 105$ MPa, SN = 2.57, $SN_1 = 1.32$						$W_T = 10.0 \times 10^6$, $M_R = 63$ MPa, SN = 4.25, $SN_1 = 2.63$					
Point i	$A_{i,i+1}$	A_{i+1}	S_{i+1}	G_{i+1}	T_{i+1} (years)	D_{i+1} (cm)	$A_{i,i+1}$	A_{i+1}	S_{i+1}	G_{i+1}	T_{i+1} (years)	D_{i+1} (cm)
1	N/A	0.00	N/A	N/A	0.00	0.00	N/A	0.00	N/A	N/A	0.00	0.00
2	21.24	21.24	0.08	1.19	8.46	6.52	3.82	3.82	0.02	1.04	1.70	3.38
3	6.46	27.70	0.12	1.27	12.24	9.23	6.16	9.98	0.05	1.13	5.22	8.75
4	3.40	31.10	0.15	1.32	15.18	10.92	5.70	15.68	0.10	1.22	9.78	15.08
5	1.65	32.75	0.18	1.36	17.68	12.08	3.89	19.57	0.15	1.31	14.97	21.30
6	0.49	33.24	0.20	1.40	20.00	12.80	1.26	20.83	0.20	1.40	20.00	23.87

SN = structural number of the original pavement structure,

S_{i+1} = strength reduction factor required at T_{i+1} service time,

D_{i+1} = overlay thickness required at T_{i+1} service time in inches,

a_1 = relative strength coefficient of asphaltic overlay material, and

G_{i+1} = traffic growth adjustment factor at T_{i+1} service time.

Table III provides sample results for two different pavement design examples. A high stability asphaltic overlay material ($a_1 = 0.44$) and crushed limestone base with 40,000 psi resilient modulus are assumed. The SN associated with the asphaltic layer is determined from Eq. (1) using the base modulus value with other design input parameters as used in the previously provided sample presentation. The estimated strength reduction factors (S_{i+1}) are assumed to vary linearly with service time. The sample overlay thickness is estimated using Eqs. (8a) and (8b) based on the relevant performance curves. The presented rehabilitation schedules offer the pavement engineer a wide range of overlay scheduling options. In these examples, there are five options available to consider as part of a complete life-cycle analysis. The estimated overlay thickness seem to agree well with practically used values.

However, it should be noted that overlay thickness associated with advanced service times are usually applied in combination with other applicable treatment actions such as milling or complete removal of the existing asphaltic surface layer. The estimated overlay design thickness is applied as a plain overlay up to a certain age that doesn't typically exceed 8–10 years. At other advanced service times (10–15 years), skin patch is applied which requires milling a portion of the asphaltic surface. The required overlay thickness is typically reduced based on the ratio of 1.0 cm with milling equivalent to 1.5 cm of plain overlay thickness. For service times greater than 15 years, reconstruction is required.

Pavement Life-cycle Analysis

The generated rehabilitation schedules in terms of timed overlay thickness provide an effective means to perform a complete life-cycle analysis. The pavement engineer can establish potential long-term rehabilitation alternatives by selecting different overlay schedules. There are several selections available but the ones with high potentials might be few. The pavement long-term performance and cost must be considered in a complete life-cycle analysis. Comparing long-term performance of potential rehabilitation alternatives is achieved through the use of the earlier presented relative performance indicator. Once the engineer decides on the overlay schedule to be used in a particular rehabilitation alternative, the related performance curves can be used to calculate the corresponding relative performance values. The required performance curves can be the same ones associated with the original pavement structure or generated in a similar way based on a new set of pavement design parameters representing the proposed conditions.

A sample presentation to evaluate long-term pavement performance is shown in Figure 5. The estimation of the indicated asphalt concrete overlay thickness ($D_{AC} = D_{i+1}$) is consistent with the data presented in Table III for the case of one million 80 kN ESAL applications but with minor variations in the timing schedule to provide for a compatible analysis period of 40 years. Overlay thickness for timing schedules not presented in the table can be estimated using linear interpolations. Three potential long-term rehabilitation alternatives have been considered in this presentation as shown in Figure 5.

The performance curve that is repeated twice in Figure 5(a) is the same one shown as Curve A in Figure 2. The corresponding rehabilitation plan consists of only one cycle of reconstruction which includes as a minimum complete removal of the existing asphalt surface layer, placement of a leveling aggregate base layer, and an application of 12.80 cm of new asphaltic pavement material. The resulting relative performance value is 0.665, which is the smallest amongst the three values

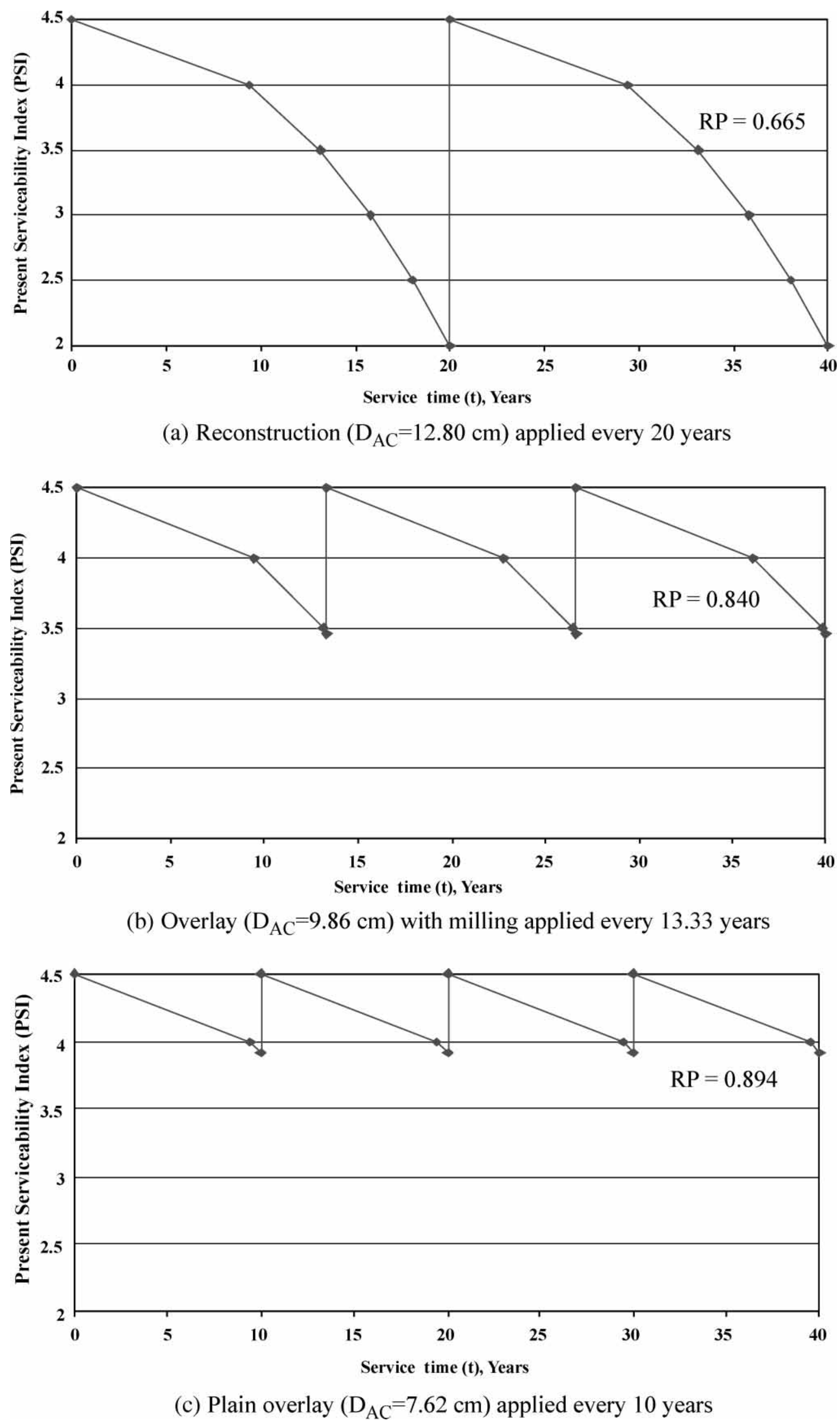

FIGURE 5 Pavement life-cycle analysis using three rehabilitation alternatives. (a) Reconstruction ($D_{AC} = 12.80$ cm) applied every 20 years. (b) Overlay ($D_{AC} = 9.86$ cm) with milling applied every 13.33 years. (c) Plain overlay ($D_{AC} = 7.62$ cm) applied every 10 years.

obtained for the three different rehabilitation alternatives. The performance curve associated with the reconstructed pavement is obtained for a design load of 1.5 million applications due to traffic increases, but since the corresponding SN has already been adjusted for these increases, the curve trend remained very much the same.

The other two rehabilitation alternatives shown in Figures 5(b),(c) are associated with two and three cycles of overlay scheduled at incremental service times of 13.33 and 10 years, respectively. The plain overlay ($D_{AC} = 7.62$ cm) applied at 10 years may not be combined with any additional treatment whereas the overlay

($D_{AC} = 9.86\,\text{cm}$) applied at 13.33 years may require milling of a portion of the existing asphaltic layer. The performance curves used in these figures are replication of the ones used in Figure 5(a), except for only the curve segments associated with the corresponding incremental service times. The rehabilitation alternative associated with Figure 5(c) has superior relative performance value ($\text{RP} = 0.894$) in comparison to the one represented by Figure 5(b). Similarly, the rehabilitation alternative associated with Figure 5(b) has superior relative performance ($\text{RP} = 0.840$) when compared to that of Figure 5(a).

In the construction of the performance curves provided in Figures 5(b),(c), it is assumed that the applied asphaltic overlay plus combined treatment will improve the pavement condition from its PSI to a new value as defined by the initial serviceability index ($P_o = 4.5$). It is further assumed that the performance trend for the rehabilitated pavement would be similar to that estimated for the original pavement. These assumptions can be revised if felt necessary by requiring a lower initial serviceability index which would result in a modified performance curve that would be associated with a lower relative performance value.

A final selection amongst the three long-term rehabilitation alternatives is subject to a detailed life-cycle cost analysis. The cost items should include initial construction, rehabilitation cycles, routine maintenance and added user cost. The initial construction and rehabilitation costs of the three alternatives can be estimated based on prevailing market prices. The differences in rehabilitation costs amongst the three alternatives are expected to be small when compared to the differences resulting from routine maintenance and added user costs. Routine maintenance and added user costs are inversely related to each other and they are both inversely related to the frequency of applied rehabilitation cycles. Routine maintenance includes repair works such as crack sealing and pothole patching necessary to maintain safe road operating conditions. Added user cost includes excess travel time cost and more importantly, added vehicle operating cost and construction delays cost. Construction delays cost is definitely much higher for a reconstruction cycle when compared to an overlay one, as reconstruction time is much longer. Therefore, the rehabilitation alternative represented by Figure 5(c) is definitely associated with lower routine maintenance and added user costs when compared to that of Figure 5(b). Similarly, these costs associated with Figure 5(b) are lower than the corresponding values for Figure 5(a).

Selection of the best (optimum) rehabilitation alternative must take into consideration both life-cycle performance and life-cycle cost (Abaza, 2002). Pavement life-cycle cost calculations are beyond the scope of this paper, but relevant detailed procedures can be found in the cited reference. However, it is most likely that the rehabilitation alternative associated with Figure 5(c) represents the best plan for it provides the highest life-cycle relative performance ($\text{RP} = 0.894$) which probably comes at the lowest life-cycle cost. This sample presentation is in support of the famous theme 'Your Choice: Bad Roads at High Cost or Good Roads at Low Cost' (Sheflin, 1983).

Pavement Management Applications

The pavement management process requires feedback on the condition of the pavement system to be effective. Feedback is generally obtained through surveys of pavement distress condition as estimated from visual inspection and simple related field measurements. The results of condition surveys are then converted into a pavement condition rating such as the pavement condition index (PCI), which applies a scale of 100 points. Unfortunately, while many highway agencies have instituted a pavement condition survey program, many others have not started such a program due to several factors. Amongst these factors are lack of strong commitments, inadequate resources, not being able to effectively utilize the feedback, and occasionally, lack of reliability in the collected data.

The approach presented in this paper provides an estimation of the future pavement condition in terms of the PSI. The PSI has been used before in modeling pavement management (Pedigo *et al.*, 1982; Abaza and Ashur, 1999). Each pavement structure can be represented by its unique performance curve. Predicted PSI data can be generated from individual performance curves or equations as provided in Figure 2, and stored as part of the inventory databank and can be easily retrieved in modeling pavement management. It is believed that the predicted PSI data provides adequate and reliable means for estimating future pavement conditions especially in the lack of periodic field condition surveys. Integration of the presented performance prediction model into the currently used pavement management systems can be easily accomplished. Representation of all different pavement structures and loading conditions in the management process can add substantial improvements to the derived solutions.

Another important application is the ability to derive estimates of the transition probabilities used in Markovian models to predict future pavement conditions. Several pavement management models have used Markovian processes to predict pavement performance (Way *et al.*, 1982; Butt *et al.*, 1987; Abaza and Ashur, 1999). All these cited pavement management models have made the assumption that any row in the transition matrix contains only the two transition probabilities ($P_{i,i}$) and ($P_{i,i+1}$). This means that a pavement section in state i can only stay in the same state or transit to the next lower one, $i+1$, during one transition. State i is defined in this paper using a specific PSI value. The time interval (d) between two transitions is typically chosen as one or two years. The transition probability ($P_{i,i+1}$) is the probability that a pavement section will transit from its present state i to

TABLE IV Sample estimation of pavement transition probabilities

	Example 1			Example 2		
	$W_T = 1.0 \times 10^6$, $M_R = 42$ MPa, SN = 3.5			$W_T = 10.0 \times 10^6$, $M_R = 63$ MPa, SN = 4.25		
		Transition probability $P_{i,i+1}$			Transition probability $P_{i,i+1}$	
Point i	$\Delta T_{i,i+1}$ (years)	$d = 1$ year	$d = 2$ years	$\Delta T_{i,I+1}$ (years)	$d = 1$ year	$d = 2$ years
1	N/A	N/A	N/A	N/A	N/A	N/A
2	3.48	0.287	0.575	1.70	0.588	1.000
3	4.12	0.243	0.485	3.52	0.284	0.568
4	4.15	0.241	0.482	4.56	0.219	0.438
5	4.09	0.244	0.489	5.16	0.194	0.388
6	4.16	0.240	0.481	5.06	0.198	0.395

state $i+1$ during one time interval (d) provided that the estimated time the section is staying between the two states is the incremental service time interval ($\Delta T_{i,i+1}$) obtained from the constructed pavement performance curve. This can be mathematically stated in Eq. (9).

$$P_{i,i+1} = \frac{d}{\Delta T_{i,i+1}} = \frac{d}{T_{i+1} - T_i} \leq 1.0 \qquad (9)$$

$$P_{i,i} = 1 - P_{i,i+1}$$

Sample calculations for two different pavement design examples are provided in Table IV. The calculations are made for two different time intervals between transitions (i.e. $d = 1$ and 2 years). In general, the values of the estimated transition probabilities appear to agree well with practically expected ones for the specified conditions. Comparison of these estimated values against corresponding ones derived from field observations is beyond the scope of this paper.

CONCLUSIONS AND RECOMMENDATIONS

It is believed that the presented deterministic model to generate individual performance curves for various pavement structures provides an effective and convenient means for pavement engineers to address many aspects of the pavement design and rehabilitation issues facing them. Evaluation of potential pavement design and rehabilitation alternatives can effectively be made using the presented techniques and procedures. It is also recommended that each new pavement design can be supplemented by its unique performance curve and rehabilitation schedule (i.e. overlay thickness schedule) similar to an automobile maintenance schedule.

It is further believed that the generated performance curves can have direct applications to many aspects of the pavement management science. The vast majority of the available pavement management systems incorporating performance prediction models require extensive historical pavement performance records to be effective, a requirement that has kept a large number of highway agencies from instituting an effective pavement management system. Each unique pavement structure can now be represented by its unique performance curve. This would provide a reasonable estimate of the future pavement condition according to the AASHTO serviceability concept as represented by the PSI. The generated performance curves can especially be effective when the intent of pavement management is establishing general solution guidelines considering the network level. Field review of pavement distress condition on the project level is practically always required prior to recommending a final rehabilitation plan.

It is, therefore, proposed to design a pavement management system that would use the performance trend data corresponding to all different pavement structures in any given road network. The system would then use that data to generate optimum rehabilitation schedules for a selected analysis period by applying an appropriate decision-making policy. The decision-making policy aims at maximizing the overall network PSI based on specified potential rehabilitation strategies and subjected to budget constraints. The proposed system would be potentially useful for professional pavement engineers and can be used as an effective teaching tool for pavement management students.

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