Performance-Based Models for Flexible Pavement Structural Overlay Design

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Abstract: Performance of flexible pavement has long been recognized as an important parameter in the design of flexible pavements. Pavement surface condition evaluated using visual inspection is periodically done to assess pavement performance over time. A distinct performance curve is then constructed for each pavement structure that relates the pavement surface condition to service time or accumulated 80 kN equivalent single axle load applications. The presented flexible pavement overlay design models are constructed using performance curve parameters to provide an adequate overlay thickness at any given future time. The undertaken approach attempts to compensate an existing pavement structure for the loss in performance (strength) that it has endured over a specified service time. In essence, this approach is similar to the mechanistic methods of overlay design that make a compensation for the loss in a particular strength indicator such as the commonly used deflection method. Therefore, compensation is made for the loss in performance as represented by appropriately selected performance curve parameters. Performance parameters are then converted into equivalent relative strength indicators, which are in turn converted into equivalent overlay thicknesses. The relative strength indicators deployed in this paper are the structural number and gravel equivalent used by the American Association of State Highway and Transportation Officials and the California Department of Transportation design methods of flexible pavement, respectively.

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Introduction

Currently, there are two design methods commonly used in practice to estimate the required overlay thickness for an existing flexible pavement structure (Yoder and Witczak 1975; Oglesby and Hicks 1982; Haung 1993). The first method is based on deflection measurement while the second one depends on distress assessment. These two methods attempt to compensate the existing pavement structure for the loss in strength it has endured over time. The first method is a mechanistic one which depends on surface deflection measurements of pavement. Pavement deflection is usually measured using especially designed expensive instruments such as the Dynaflect or falling weight deflectometer (FWD). Deflection measurements, traffic data, and existing pavement properties are then used to estimate the required overlay thickness in what is known as backward solution or backcalculation of the multilayer linear elastic theory. The elastic method has been simplified by some professional agencies such as the Asphalt Institute and California Department of Transportation using design charts (Caltrans 1995; AI 1996). The deflection method is relatively expensive to perform as local governments often hire private consultants to perform the related study for each project, and the deflection technology is not available in most developing countries.

The second method of overlay design, known as the effective thickness approach or component analysis method, depends on conducting pavement condition evaluation with the outcome translated into equivalency conversion factors recommended by the Asphalt Institute and American Association of State Highway and Transportation Officials (AASHTO) overlay design methods (AASHTO 1993; AI 1996). The equivalency conversion factors account for the degree of distress presents in the existing pavement structure when compared to a new pavement. For example, 1.0 cm of an existing asphalt concrete thickness is generally equivalent to 0.3–0.7 cm of a new asphalt concrete pavement. Essentially, this method requires designing a new pavement structure using the layers' thicknesses and properties of the existing pavement structure. The required overlay thickness is then estimated from the difference between the thickness of newly designed full-depth asphalt pavement and equivalent thickness of the existing pavement structure obtained from multiplying each layer thickness by its corresponding equivalency factor. This method is considered sensitive to the value of design equivalency factor, therefore, good judgment and well defined guidelines are needed in determining the design equivalency factors. A third method known as the "prescription procedure" is also commonly used by some local governments where predetermined overlay thicknesses and rehabilitation schedules are already established for various road classes based on experience and engineering judgment.

The proposed method of overlay design depends on using the pavement performance curve as the main design input parameter. The area falling under the pavement performance curve is a direct measure of pavement design strength (Yoder and Witczak 1975;

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Haung 1993), and it has been used as a basis for yielding an optimum pavement design (Abaza and Abu-Eisheh 2003). The proposed method attempts to compensate the existing pavement structure for design strength loss using appropriate strength indicators derived from the performance curve such as the area falling under the utilized curve portion. In essence, this is a way to compensate the pavement structure for loss in strength it has endured over the past service period. The pavement design strength is defined using an appropriate strength indicator such as the relative strength indicator used in the empirical design methods of flexible pavement. The American Association of State Highway and Transportation Officials (AASHTO 1993) method and the California Department of Transportation (Caltrans 1995) method are two popular examples among these methods.

The pavement performance curves can be generated using stochastic prediction models (Shahin et al. 1987; George et al. 1989; Gopinath et al. 1994) or developed from historical records of pavement distress. A convenient and practical prediction model to generating the performance curve for a particular pavement structure was developed using the AASHTO design method of flexible pavement (Abaza 2002; Abaza and Abu-Eisheh 2003). This model is based on performing an incremental analysis of the AASHTO basic design equation of flexible pavement (AASHTO 1993), and it can be used in the absence of historical pavement distress records. An overview of the AASHTO performance prediction model is provided in a separate section.

Overview of Main Overlay Design Methods

Overlay design methods of flexible pavement are generally two types: the effective thickness method and nondestructive testing method. The related design procedures are similar to the procedures used for designing a new pavement structure except that the condition or remaining life of the existing pavement structure at the time of the overlay is taken into consideration (Haung 1993). Brief overviews of the most popular methods of overlay design are provided in this section.

Asphalt Institute Method

The Asphalt Institute provides two design methods for the design of asphalt overlay on asphalt pavement (AI 1996). The first method is the effective thickness method that estimates the overlay thickness as the difference between the thickness required for a new full-depth asphalt pavement and the effective thickness of the existing pavement as provided in the following equation:

$$h_{\mathrm{OL}} = h_n - h_e = h_n - \sum_{i=1}^{n} C_i h_i \tag{1}$$

where h_{OL}=required asphalt overlay thickness; h_n=thickness of new full-depth asphalt pavement; h_e=effective thickness of the existing pavement; h_i=thickness of the ith layer of the existing pavement; C_i=conversion factor associated with the ith existing layer; and n=number of layers in the existing pavement structure.

The conversion factor is used to account for the pavement distress condition and it is estimated based on material classification and description (AI 1996). This method is simple to apply, however, the estimated overlay thickness is very sensitive to the used design conversion factors. The second method is based on deflection measurements taken using the Benkelman beam, projected overlay traffic, temperature adjustment factor, and critical period adjustment factor. The design overlay thickness is obtained from a design chart using overlay traffic and a design deflection indicator called the representative rebound deflection.

American Association of State Highway and Transportation Officials Method

The AASHTO method of overlay design is based on the remaining life concept and requires nondestructive testing to estimate the in situ moduli and structural capacity of subgrade and various pavement layers. For an overlaid pavement, the structural capacity of the original pavement, defined using the structural number, is a function of the loads applied before overlay as well as those applied after overlay. The following equation is the basic design equation used to determine the asphalt overlay thickness on asphalt pavement (AASHTO 1993):

$$h_{\mathrm{OL}} = \frac{\mathrm{SN}_{\mathrm{OL}}}{a_{\mathrm{OL}}} = \frac{\mathrm{SN}_y - F_{\mathrm{RL}}\mathrm{SN}_{\mathrm{eff}}}{a_{\mathrm{OL}}} \tag{2}$$

where h_{OL}=required thickness of asphalt overlay; $\mathrm{SN}_{\mathrm{OL}}$=required structural number of asphalt overlay; a_{OL}=structural layer coefficient of asphalt overlay; SN_y=total structural number required to support the overlay traffic over existing subgrade conditions; F_{RL}=remaining life factor accounting for damage of the existing pavement as well as the desired degree of damage to the overlay at the end of overlay traffic; and $\mathrm{SN}_{\mathrm{eff}}$=total effective structural number of the existing pavement prior to overlay.

The remaining life factor (F_{RL}) is an adjustment factor applied to the effective structural number of the existing pavement to reflect a more realistic assessment of the weighted effective strength during the overlay period. The AASHTO provides five different methods from which different estimates of the remaining life factor are obtained. Elliot (1989) revealed some inconsistencies in the overlay design thickness in relation to the remaining life factor and recommended its exclusion from Eq. (2) while Fwa (1991) proposed a new expression for estimating the remaining life factor. Similarly, AASHTO proposes three different methods to estimate the total effective structural number associated with the existing pavement structure. One of these methods, called the nondestructive testing (NDT) method, is based on the nondestructive testing (deflections) measurement and interpretation. Hoffman (2003) proposed a direct and simple method for estimating the effective structural number of the existing pavement, called the YONAPAVE method, which is based on the interpretation of the measured FWD deflection basins using mechanistic and practical approaches. Hall et al. (1992) suggested other revisions to the AASHTO overlay design procedures to make them easier to understand and use, and more adaptable to calibration by local agencies. Therefore, it appears that the AASHTO method has certain inconsistencies and difficulties, which restricts its use especially by local governments.

State Department of Transportation Methods

Several state Department of Transportations have developed their own flexible overlay design models using mechanistic–empirical procedures (Mamlouk et al. 1990; Zhou et al. 1992; Caltrans 1995; Pierce and Mahoney 1996). These procedures apply the deflection-based NDT to develop models that are calibrated to meet the local conditions of each state highway system. Therefore, the use of these models is restricted to the states that developed them. Also, Maestas and Mamlouk (1992) evaluated four different computer software programs that apply the concept of

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backcalculation and determined that although the four programs are characteristically different, neither layer modulus nor overlay thickness results were statistically different in most cases, however, from a practical perspective backcalculation, as represented by these programs, is not adequate for use by the practitioner.

Methodology

The proposed performance-based models for estimating the overlay thickness for a particular pavement structure are empirical models derived from the constructed pavement performance curve. The area falling under the performance curve is a direct measure of pavement design strength (Yoder and Witczak 1975; Haung 1993). The AASHTO method relies mainly on the performance curve to define the expected pavement serviceability in relation to the 80 kN equivalent single axial load (ESAL) applications. The AASHTO empirical performance equations, derived from the AASHO road test, show that the trend of the performance curve is directly related to the pavement design strength as indicated by the design structural number (AASHTO 1993). Recently published work has used the area under the performance curve as a measure of pavement strength to develop optimum design procedure and perform optimum life-cycle analysis of flexible pavement (Abaza 2002; Abaza and Abu-Eisheh 2003).

The first proposed model is based on the area falling under the utilized performance curve portion. Two other models are proposed using two main performance parameters derived from the generated performance curve. These two main performance parameters are the pavement condition indicator and accumulated 80 kN equivalent single axle load applications, which are used to define the trend of the performance curve. The three proposed models are time dependent and they can estimate the required overlay thickness for a particular pavement structure at any given future time. The proposed models are applied to two very popular flexible pavement design methods that use relative strength design indicators, namely, the AASHTO and Caltrans design methods. The three models are described in detail in the subsequent subsections.

Utilized Performance Area Overlay Model

The first proposed model uses the area falling under the utilized portion of the performance curve as the main parameter to estimate the overlay thickness as provided in Eq. (3). The model proposed by the following equation attempts to compensate the pavement structure for the loss in strength it has endured over time using the ratio of the area under the utilized curve portion to the total area falling under the performance curve. Fig. 1 shows a typical pavement performance curve with the total area falling under the curve is being designated (A_0). Fig. 2 shows the area under the utilized portion of the performance curve $A(t)$ as a function of service time. The performance area ratio is raised to power (n_1) to be determined based on experience and engineering judgment. The selected value of (n_1) provides an overlay model that will generate overlay thicknesses similar to the values used in practice

$$S(t) = \left(\frac{A(t)}{A_0}\right)^{n_1} \times S_o \times F_g(t) \qquad (3)$$

where

$$0.0 \leqslant S(t) \leqslant S_o \times F_g(t)$$

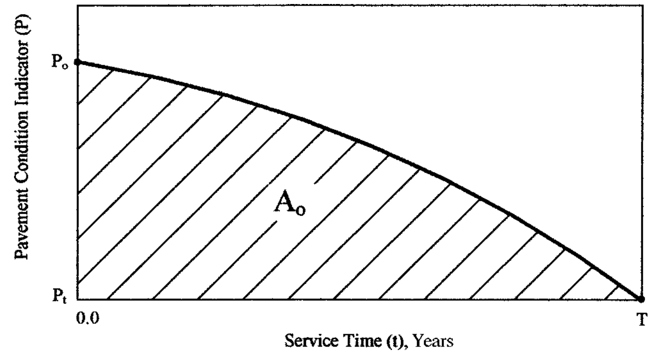

Fig. 1. Typical pavement performance curve

$$0.0 \leqslant A(t) \leqslant A_0$$

$$0.0 \leqslant t \leqslant T$$

The proposed model estimates the present strength $S(t)$, associated with the required overlay thickness at a given future time (t) as a proportion of the initial design strength provided by the original pavement structure (S_o). The estimated present overlay strength can then be converted into an equivalent overlay thickness using an appropriate relative strength coefficient as outlined in a subsequent subsection. A traffic growth factor $F_g(t)$ can be applied to Eq. (3) to account for traffic growth that wasn't anticipated in the original design. The proposed model can be used for any future time less than or equal to the design service life (T) of the original pavement structure.

Pavement Condition Indicator Overlay Model

The second proposed model is constructed using the pavement condition indicator $P(t)$ at a given future time (t) as the main performance parameter. Fig. 3 shows a typical pavement performance curve defined as a function of the pavement condition indicator and service time. The ratio of the change in the pavement condition indicator $\Delta P(t)$ at a given future time to the difference in the initial and terminal condition indicator values (ΔP) is used as the main parameter in the model proposed by the following equation with this ratio raised to power (n_2). The ratio used will yield present overlay strength $S(t)$ values that range between minimum of zero and maximum value that equals to the initial design strength (S_o) provided by the original pavement structure

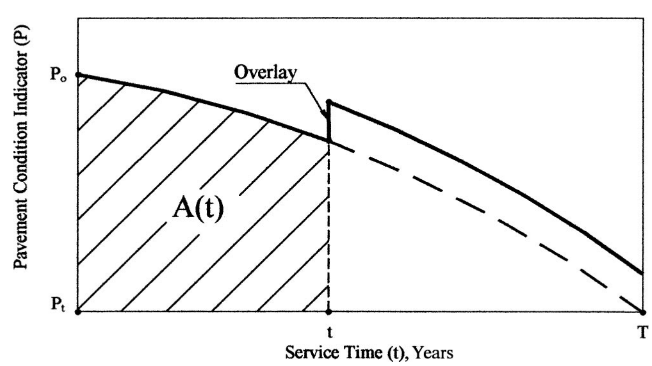

Fig. 2. Typical overlay model based on utilized performance area

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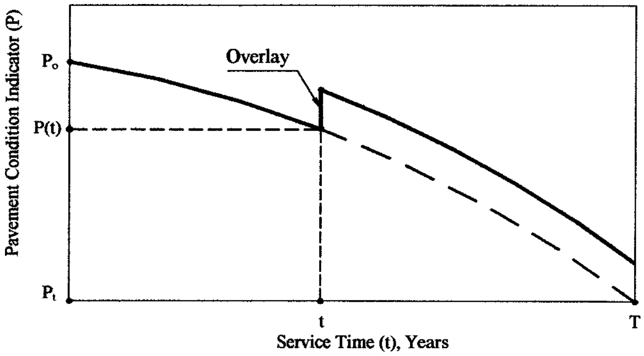

Fig. 3. Typical overlay model based on pavement condition indicator

$$S(t) = \left(\frac{\Delta P(t)}{\Delta P}\right)^{n_2} \times S_o \times F_g(t) \tag{4}$$

where

$$0.0 \leqslant S(t) \leqslant S_o \times F_g(t)$$

$$\Delta P(t) = P_o - P(t); \quad \Delta P = P_o - P_t$$

$$P_t \leqslant P(t) \leqslant P_o$$

$$0.0 \leqslant t \leqslant T$$

The initial and terminal pavement condition indicator values are designated (P_o) and (P_t), respectively. Other variables as defined earlier. Again, the power (n_2) needs to be estimated based on experience and judgment as will be demonstrated in the presented sample results.

Accumulated Load Applications Overlay Model

The third proposed model deploys the accumulated 80 kN ESAL applications, estimated from the generated pavement performance curve at a given future time, as the main performance parameter. Fig. 4 shows a typical performance curve defined using the pavement condition indicator $P(t)$ and the accumulated 80 kN equivalent single axle load applications $N(t)$. The ratio of the accumulated 80 kN equivalent single axle load applications $N(t)$ to the total 80 kN ESAL applications (N_T) associated with the original pavement design is used as the main parameter in the model proposed by Eq. (5) with this ratio raised to power (n_3)

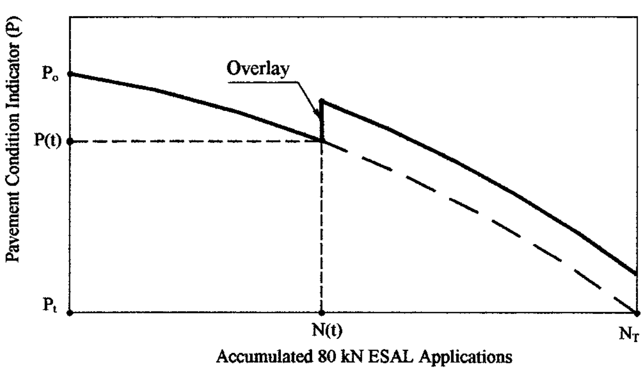

Fig. 4. Typical overlay model based on accumulated load applications

$$S(t) = \left(\frac{N(t)}{N_T}\right)^{n_3} \times S_o \times F_g(t) \tag{5}$$

where

$$0.0 \leqslant S(t) \leqslant S_o \times F_g(t)$$

$$0.0 \leqslant N(t) \leqslant N_T$$

$$0.0 \leqslant t \leqslant T$$

According to Eq. (5), the estimated present overlay strength $S(t)$ again ranges between minimum of zero and maximum value that is equal to the design strength associated with the original pavement structure with the possible inclusion of a traffic growth factor. The traffic growth factor $F_g(t)$ can be estimated using the Asphalt Institute recommended formula as provided in the following equation (AI 1991):

$$F_g(t) = \frac{(1+r)^t - 1}{r \times t} \tag{6}$$

The estimated traffic growth factor is based on an annual traffic growth rate (r) in decimal form for a projected time period (t). Therefore, it is assumed that the traffic growth factor will account for future increase in traffic loads for a period of time that is equal to the time used in the design of the overlay thickness.

Asphalt Overlay Thickness Design

The present overlay strength estimated from the three proposed models can be converted into an equivalent asphalt overlay design thickness using an appropriate asphalt relative strength coefficient. The required overlay thickness $D(t)$ is defined as the ratio of the present overlay strength $S(t)$ to the asphalt relative strength coefficient (C_s) as provided in Eq. (7). This approach of converting design strength into an equivalent layer thickness has been used by two popular flexible pavement design methods, namely, the AASHTO and Caltrans methods (AASHTO 1993; Caltrans 1995). The AASTHO design method uses the asphalt layer relative strength coefficient designated (a_1) while Caltrans uses a similar coefficient known as the asphalt gravel equivalent factor (G_{f_1}).

$$D(t) = \frac{S(t)}{C_s} = \frac{S(t)}{a_1} = \frac{S(t)}{G_{f_1}} \tag{7}$$

Application of Eq. (7) for estimating the asphalt overlay thickness $D(t)$ at any given future time is performed in conjunction with using one of the two mentioned design methods of flexible pavement, namely, AASHTO and Caltrans. Brief overviews of these two methods in relation to overlay thickness estimation are presented in the two subsequent subsections.

Overlay Thickness Design Using American Association of State Highway and Transportation Officials Method

The AASHTO design method of flexible pavement uses a relative strength design indicator known as the structural number (SN). The structural number is derived from the AASHTO basic design equation based on design load applications, materials strength, and traffic and performance prediction parameters (AASHTO 1993). The structural number is then converted into pavement layers' thicknesses (D_j) using layers' relative strength coefficients

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(a_j) as provided in the following equation with (m) representing the number of pavement layers. The pavement layers' relative strength coefficients (a_j) have been related to commonly used materials strength indicators such as the resilient modulus (M_R), resistance value (R), and California bearing ratio. Correlation charts can be consulted for the purpose of making the needed conversion (Yoder and Witczak 1975; Haung 1993).

$$\mathrm{SN} = \sum_{j=1}^{m} a_j \times D_j \tag{8}$$

$$\mathrm{SN}_1 = a_1 \times D_1$$

The proposed performance-based models estimate the present overlay strength $S(t)$ in relation to the design strength of the original pavement structure (S_o). According to the AASHTO design method, the design strength of the original pavement structure is the structural number (SN_o). Therefore, the structural number can be used in Eqs. (3)–(5) to replace the design strength of the original pavement structure. The structural number of the original asphalt layer is (SN_1). Eq. (7) used for estimating the required overlay thickness $D(t)$ becomes Eq. (9) when the present overlay strength $S(t)$ is replaced by the present structural number $\mathrm{SN}(t)$

$$D(t) = \frac{S(t)}{C_s} = \frac{\mathrm{SN}(t)}{a_1} \tag{9a}$$

$$\mathrm{SN}(t) = \left(\frac{A(t)}{A_0}\right)^{n_1} \times \mathrm{SN}_o \times F_g(t) \tag{9b}$$

or

$$\mathrm{SN}(t) = \left(\frac{\Delta P(t)}{\Delta P}\right)^{n_2} \times \mathrm{SN}_o \times F_g(t) \tag{9c}$$

or

$$\mathrm{SN}(t) = \left(\frac{N(t)}{N_T}\right)^{n_3} \times \mathrm{SN}_o \times F_g(t) \tag{9d}$$

The AASHTO basic design equation for estimating the required design SN is provided in a subsequent section entitled "AASHTO Peformance Prediction Model Overview." AASHTO uses the present serviceability index (PSI) as a measure of pavement performance. Therefore, the change in the pavement condition indicator $\Delta P(t)$ becomes the change in the PSI $\Delta \mathrm{PSI}(t)$. The structural number (SN_o) in Eq. (9) can be replaced by the original asphalt layer structural number (SN_1) if deterioration is mainly assumed to affect the surface layer.

Overlay Thickness Design Using Caltrans Method

The Caltrans design method of flexible pavement is another empirical design method that uses a relative strength indicator known as the gravel equivalent (GE). The GE has been related to two main design parameters, namely; traffic loads and materials strength as given by Eq. (10) (Caltrans 1995). The traffic loading condition is represented by the traffic index (TI), which is related to the design 80 kN ESAL applications (N_T) as provided in the following equation:

$$\mathrm{GE} = 0.0032 \times \mathrm{TI} \times (100 - R) \tag{10}$$

$$\mathrm{GE}_1 = 0.0032 \times \mathrm{TI} \times (100 - R_b)$$

where

$$\mathrm{TI} = 9.0 \times \left(\frac{N_T}{10^6}\right)^{0.119}$$

Materials strength is defined using the resistance value with the subgrade and base resistance values designated (R) and (R_b), respectively. Then, the GE required for a particular pavement structure is converted into layers' thicknesses using the layer gravel equivalent factor (G_{f_j}) as provided in the followng equation with (m) representing the number of pavement layers. The gravel equivalent factor for asphalt concrete pavement depends, according to Caltrans method, on the traffic loading condition as represented by the traffic index. The gravel equivalent for the original asphalt layer is (GE_1)

$$\mathrm{GE} = \sum_{j=1}^{m} G_{f_j} \times D_j \tag{11}$$

$$\mathrm{GE}_1 = G_{f_1} \times D_1$$

The proposed performance-based models estimate the present overlay strength $S(t)$ in relation to the design strength of the original pavement structure (S_o). According to the Caltrans design method, the design strength of the original pavement structure is the gravel equivalent (GE_o). Therefore, the gravel equivalent can be used in Eqs. (3)–(5) to replace the design strength of the original pavement structure. Eq. (7) used for estimating the required overlay thickness $D(t)$ becomes the following equation when the present overlay strength $S(t)$ is replaced by the present gravel equivalent $\mathrm{GE}(t)$:

$$D(t) = \frac{S(t)}{C_s} = \frac{\mathrm{GE}(t)}{G_{f_1}} \tag{12a}$$

$$\mathrm{GE}(t) = \left(\frac{A(t)}{A_0}\right)^{n_1} \times \mathrm{GE}_o \times F_g(t) \tag{12b}$$

or

$$\mathrm{GE}(t) = \left(\frac{\Delta P(t)}{\Delta P}\right)^{n_2} \times \mathrm{GE}_o \times F_g(t) \tag{12c}$$

or

$$\mathrm{GE}(t) = \left(\frac{N(t)}{N_T}\right)^{n_3} \times \mathrm{GE}_o \times F_g(t) \tag{12d}$$

The asphalt gravel equivalent factor (G_{f_1}) can be obtained based on the design traffic index by consulting Caltrans design manual (Caltrans 1995). Again, the gravel equivalent (GE_o) in Eq. (12) can be replaced by the original asphalt layer gravel equivalent (GE_1) if deterioration is mainly assumed to affect the surface layer. The required performance parameters can be obtained from the AASHTO performance prediction model or estimated from historical records of pavement distress.

Advantages of Proposed Performance-Based Overlay Models

It must first be recognized that each design method applies different approach and requires different design parameters with each method yielding an overlay thickness that is very unlikely to be the same thickness obtained from another method. However, the presented performance-based overlay design models have certain advantages over the outlined AI and AASHTO overlay design methods that can be summarized as follows:

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1. The proposed performance-based overlay models require mainly the performance curve associated with the original pavement structure. It can be estimated using the presented AASHTO performance prediction model or from historical records of pavement distress that are typically obtained for pavement management. This is an advantage especially to United States local governments and developing countries that lack the resources and expertise to perform NDT testing required by the AASHTO method and other mechanistic approaches.
2. A local agency can only generate one performance curve and consequently one overlay design model for each pavement system consisting of roads with similar pavement structures and traffic conditions. Therefore, a small local government may only need to develop a limited number of overlay models.
3. The actual performance curve is expected to deviate from the trend depicted by the estimated performance curve. However, the overall impact on overlay design thickness is expected to be small since the presented overlay models apply the ratio of a present performance parameter such as the utilized area to a total performance parameter such as the total curve area.
4. The final outcome of the proposed overlay models is an equation that is only time dependent, as will be demonstrated in the sample presentation section, and it is used to estimate the overlay thickness at any given future time. It occasionally happens that an agency hires a consultant to perform the NDT testing and recommend an overlay thickness, but the construction gets delayed a couple of years or more, and thus the recommended overlay thickness is no longer valid. But by using the proposed overlay models, the overlay design thickness can easily be updated without any additional cost.
5. The proposed overlay models always yield overlay thicknesses that are reasonable without any major discrepancies. This is because the overlay thickness is estimated as a proportion of the original pavement strength. The original pavement strength can be the strength associated with the entire pavement structure if surface condition shows major signs of deformation and damage or only the strength associated with the asphalt layer if surface condition shows otherwise.
6. An estimate for the total effective structural number of the existing pavement, used by the AASHTO overlay design method, at a given service time (t) can be estimated from the performance curve based on the ratio of the unutilized curve area to total curve area, raised to power m, and multiplied by the structural number associated with the original pavement structure (SN_o) as provided in the following equation:

$$\mathrm{SN}_{\mathrm{eff}}(t) = \left(\frac{A_0 - A(t)}{A_0}\right)^m \times \mathrm{SN}_o \qquad (13)$$

7. A new performance-based overlay model can be proposed that is compatible to the AASHTO basic design equation presented in Eq. (2), which uses the total effective structural number of the existing pavement as obtained from Eq. (13). An empirical equation for overlay design thickness can then be derived that is only time-dependent.

Overlay Life-Cycle Cost Analysis Model

An overlay life-cycle cost analysis (LCCA) can be performed once a specific overlay model has been derived for a particular pavement structure (project). The presented overlay LCCA model considers a number of pavement rehabilitation cycles (N) scheduled at equal time intervals (Δd) over a selected life-cycle analysis period (D). The thickness of the required asphalt overlay is estimated from the derived overlay model and assumed to remain unchanged for each scheduled rehabilitation cycle. The rehabilitation cycle may consist of plain overlay, overlay combined with cold milling, or complete removal of the existing surface layer and replacement with new material. However, each rehabilitation cycle within the same analysis period is assumed to consist of the same treatment plan. The present value of the life-cycle cost (P_0) associated with a particular rehabilitation schedule is given by Eq. (14). A rehabilitation schedule consists of (N) overlay cycles

$$P_0 = C_0 \times \sum_{k=1}^{N} f(P/F, i, D_k) \qquad (14)$$

where

$$N = \left(\frac{D}{\Delta d} - 1\right)$$

$$D_k = k \times \Delta d \quad (k = 1, \ldots, N)$$

$$D_N = (D - \Delta d)$$

$$f(P/F, i, D_k) = \frac{1}{(1+i)^{D_k}}$$

The overlay LCCA model considers the cost of each rehabilitation cycle as a future value (F) that is converted into a present value (P) using the economical conversion factor $f(P/F, i, D_k)$, where (i) = annual discount rate and (D_k) = scheduled time (years) of the kth overlay cycle. The life-cycle analysis period (D) is selected such that the resulting number of overlay rehabilitation cycles (N) is an integer and the scheduling time of the Nth cycle is equal to ($D - \Delta d$). Since it is assumed that all overlay rehabilitation cycles scheduled within a selected analysis period consist of the same treatment plan, then the associated cost is assumed constant at an estimated rate of (C_0) in United States dollars per square meter. The overlay LCCA is performed by varying the number of selected rehabilitation cycles (N) within the same analysis period as will be demonstrated in the sample presentation. A complete pavement LCCA requires the inclusion of other cost items such as routine maintenance and added user costs (FHWA 1994; Abaza 2002).

American Association of State Highway and Transportation Officials Performance Prediction Model Overview

A procedure for generating a pavement performance curve, using an incremental analysis of the AASHTO basic design equation for flexible pavement design, has been used in pavement design and rehabilitation (Abaza 2002; Abaza and Abu-Eisheh 2003). The developed procedure provides a simple tool to predict 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 performance curve are the PSI and 80 kN ESAL applications. These two parameters are also related to materials properties, drainage and environmental conditions, and performance reliability. The design

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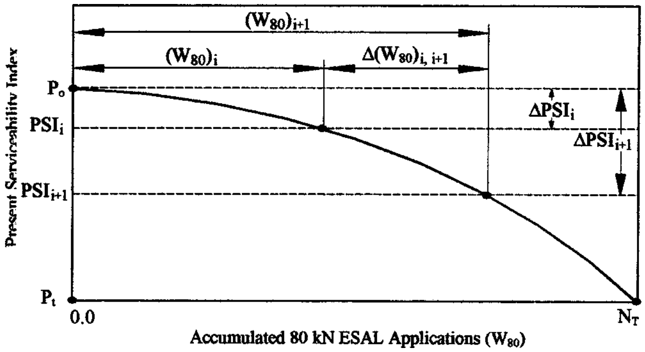

Fig. 5. Basic pavement performance curve

approach applies all related parameters to obtain a measure of the required structural strength through an index known as the SN. The following equation provides the basic equation used for the design of flexible pavement (AASHTO 1993):

$$\log W_{80} = Z_R S_o + 9.36 \log(\mathrm{SN}+1) + \frac{\log\left[\dfrac{\Delta \mathrm{PSI}}{4.2-1.5}\right]}{0.40 + \dfrac{1{,}094}{(\mathrm{SN}+1)^{5.19}}} + 2.32 \log(M_R) - 8.27 \qquad (15)$$

where W_{80} = 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 or present serviceability index (P_0) and the terminal serviceability index (P_t); SN = design structural number indicative of the total required pavement thickness; and M_R = subgrade resilient modulus and must be in pound per square inch.

In the design mode and after all related parameters are estimated, Eq. (15) is solved for the design SN by trial and error or using the equivalent AASHTO design chart (AASHTO 1993). The approach used to define a pavement performance curve as a function of the present serviceability index and 80 kN ESAL applications or service time is based on the direct use of Eq. (15). The incremental 80 kN ESAL applications $(W_{80})_i$ is calculated by specifying varying values of the incremental change in the present serviceability index ($\Delta \mathrm{PSI}_i$). The incremental change in the present serviceability index is defined as the difference between the initial serviceability index (P_0) and the incremental present serviceability index (PSI_i). The incremental present serviceability index is varied between its assigned initial and terminal values. Fig. 5 illustrates the basic concept by which the difference between two successive data points can be used to construct a pavement performance curve. The estimated incremental change in load applications ($\Delta(W_{80})_{i,i+1}$) can then be converted into an equivalent incremental service time interval ($\Delta T_{i,i+1}$) using the following equation. The assumption made in establishing the following equation is that the 80 kN ESAL applications increase linearly with time:

$$\Delta T_{i,i+1} = \frac{\Delta(W_{80})_{i,i+1}}{N_T} \times T \qquad (16)$$

where

$$\Delta(W_{80})_{i,i+1} = (W_{80})_{i+1} - (W_{80})_i, \quad i = 1,2,\ldots\ldots,n.$$

$$(W_{80})_i = F(\Delta \mathrm{PSI}_i, \mathrm{SN}, M_R, Z_R, S_o)$$

from Eq. (15)

$$(W_{80})_{i+1} = F(\Delta \mathrm{PSI}_{i+1}, \mathrm{SN}, M_R, Z_R, S_o)$$

from Eq. (15)

$$N_T = \sum_{i=1}^{n} \Delta(W_{80})_{i,i+1}$$

Note that N_T is also the total number of 80 kN ESAL applications estimated over a design life of T years

$$\mathrm{SN} = F(N_T, \Delta \mathrm{PSI}, Z_R, S_o, M_R)$$

$$T = \sum_{i=1}^{n} \Delta T_{i,i+1}$$

and

$$N_{T_{i+1}} = \sum_i \Delta(W_{80})_{i,i+1} = (W_{80})_{i+1}$$

$$N_{T_1} = 0.0$$

where $N_{T_{i+1}}$ = cumulative number of 80 kN ESAL applications estimated over a service life of T_{i+1} years. Also

$$T_{i+1} = \sum_i \Delta T_{i,i+1}$$

$$T_1 = 0.0$$

where T_{i+1} = cumulative service time in years associated with the cumulative 80 kN ESAL applications ($N_{T_{i+1}}$). In addition

$$\Delta \mathrm{PSI}_i = P_0 - \mathrm{PSI}_i$$

$$\Delta \mathrm{PSI} = P_0 - P_t$$

$$\mathrm{PSI}_i = P_0 - (i-1) \times \Delta P_{i,i+1}, \quad i = 1,2,\ldots,n+1$$

$$n = \frac{\Delta \mathrm{PSI}}{\Delta P_{i,i+1}}$$

$\Delta P_{i,i+1}$ = specified incremental change in the PSI value used to generate ($n+1$) data points to be used in the construction of a particular pavement performance curve. It must be specified either as a tenth or hundredth of a point to ensure n will be an integer. A performance curve is then constructed by plotting the incremental PSI_i versus the cumulative aging time (T_{i+1}) or cumulative 80 kN ESAL applications ($N_{T_{i+1}}$).

The total area (A_0) falling under the entire performance curve is calculated as the sum of the incremental areas ($A_{i,i+1}$) with each incremental area calculated as the area of a trapezoidal strip bounded by two curve points as provided in the following equation. The utilized performance area $A(t)$ at any given future time (t) is calculated in a similar way but using only the applicable curve points

$$A_0 = \sum_{i=1}^{n} A_{i,i+1}$$

$$A_{i,i+1} = \left[\tfrac{1}{2}(\mathrm{PSI}_i + \mathrm{PSI}_{i+1}) - P_t\right] \times \Delta T_{i,i+1} \qquad (17)$$

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Table 1. Sample Performance Curve Parameters Using American Association of State Highway and Transportation Officials Prediction Model

Point i	PSI_i	ΔPSI_i	$(W_{80})_i$ $(\times 10^6)$	$\Delta(W_{80})_{i,i+1}$ $(\times 10^6)$	$\Delta T_{i,i+1}$ (years)	$N_{T_{i+1}}$ $(\times 10^6)$	T_{i+1} (years)
1	4.5	0.0	0.00	N/A[a]	N/A[a]	0.00	0.00
2	4.0	0.5	0.60	0.60	1.20	0.60	1.20
3	3.5	1.0	1.78	1.18	2.36	1.78	3.56
4	3.0	1.5	3.36	1.58	3.16	3.36	6.72
5	2.5	2.0	5.26	1.90	3.80	5.26	10.52
6	2.0	2.5	7.45	2.19	4.38	7.45	14.90
7	1.5	3.0	10.00	2.55	5.10	10.0	20.00

Note: PSI=present serviceability index.

[a]Not available.

Table 2. Sample Overlay Thicknesses Using Utilized Performance Area Model

Time (t)[a] years	$A(t)$	$F_g(t)$	Asphalt overlay thickness $[D(t)]$, cm: AASHTO Method $n_1=1.5$	AASHTO Method $n_1=2$[b]	AASHTO Method $n_1=2.5$	Caltrans method $n_1=1.5$	Caltrans method $n_1=2$[b]	Caltrans method $n_1=2.5$
0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00
1.20	3.30	1.00	0.79	0.28	0.10	0.84	0.30	0.10
3.56	8.61	1.04	3.40	2.06	1.24	3.63	2.18	1.32
6.72	14.14	1.09	7.49	5.82	4.52	8.00	6.20	4.83
10.52	18.89	1.16	12.32	11.05	9.93	13.16	11.81	10.62
14.90	22.17	1.24	16.74	16.28	15.80	17.91	17.42	16.92
20.00	23.45	1.34	19.66	19.66	19.66	21.03	21.03	21.03

Note: AASHTO=American Association of State Highway and Transportation Officials.

[a]The same as T_{i+1} provided in Table 1.

[b]Selected as the best appropriate model.

Sample Presentation

A sample presentation for estimating overlay thickness is provided in this section using both the AASHTO and Caltrans design methods. In the presented sample overlay thickness calculations, it is assumed that the deterioration of the pavement structure mainly affects the asphalt layer. Therefore, the structural number and gravel equivalent corresponding to the surface layer (SN_1 and GE_1) are used in lieu of the structural number and gravel equivalent corresponding to the original pavement structure (SN_o and GE_o).

Sample American Association of State Highway and Transportation Officials and Caltrans Flexible Pavement Designs

The sample pavement design input parameters needed for the calculation of the original pavement surface thickness (D_1) and generation of the corresponding performance curve according to the AASHTO method are as follows:

Design 80 kN ESAL applications $(N_T)=10.0\times 10^6$
subgrade resilient modulus $(M_R)=3$ MPa (9,000 psi)
granular base resilient modulus $(M_{R_b})=280$ MPa (40,000 psi)
initial present serviceability index $(P_0)=4.5$
terminal present serviceability index $(P_t)=1.5$
95% reliability $(Z_R)=-1.645$
overall standard deviate $(S_o)=0.35$
pavement design analysis period $(T)=20$ years
incremental PSI change $(\Delta P_{i,i+1})=0.5$

The resulting design yields 4.067 structural number (SN_o) based on design subgrade modulus of 63 MPa (9,000 psi). The estimated structural number is needed for the generation of the corresponding performance curve according to the AASHTO prediction model. Then, using 280 MPa (40,000 psi) base resilient modulus results in a 2.542 surface structural number (SN_1), which corresponds to a pavement surface thickness of 14.68 cm (5.78 in.) assuming high stability asphalt mix ($a_1=0.44$). The structural numbers (SN_o and SN_1) are obtained using Eq. (15).

The pavement design input parameters needed for estimating the asphalt layer thickness according to the Caltrans method are the same design 80 kN ESAL applications ($N_T=10.0\times 10^6$), and 78 granular base resistance value (R_b) which is equivalent to 280 MPa (40,000 psi) resilient modulus value. The resulting TI value is 12, which corresponds to 1.64 gravel equivalent factor (G_{f_1}) for asphalt mix type B (Caltrans 1995). The calculated gravel equivalent for asphalt (GE_1) is 25.76 cm (0.845 ft) resulting in an equivalent layer thickness of 15.70 cm (6.18 in.).

Sample Overlay Design Thickness Models

The performance parameters needed for the construction of the corresponding performance curve according to the presented AASHTO prediction model are summarized in Table 1. The resulting total area falling under the curve (A_0) is 23.45. The estimated sample asphalt overlay thicknesses using the utilized performance area model are provided in Table 2 for both AASHTO and Caltrans design methods. These sample results are obtained using Eq. (9*b*) for AASHTO with (SN_o) replaced by (SN_1) and Eq. (12*b*) for Caltrans with (GE_o) replaced by (GE_1). Three different values have been tested for the model power (n_1). The thickness values corresponding to a power value of 2.0 seem to be more appropriate compared to the others according to the author's subjective assessment. The power value of 2.5 is also acceptable, but the value of 1.5 provides overlay thicknesses that are somewhat higher than what typically used in practice, especially at or before the pavement middle age. The effect of power value (n_1) on overlay thicknesses diminishes at advanced service times as shown in Fig. 6 using results from the AASHTO method. The power value is generally selected based on experience and engineering judgment, and by comparing the resulting overlay thick-

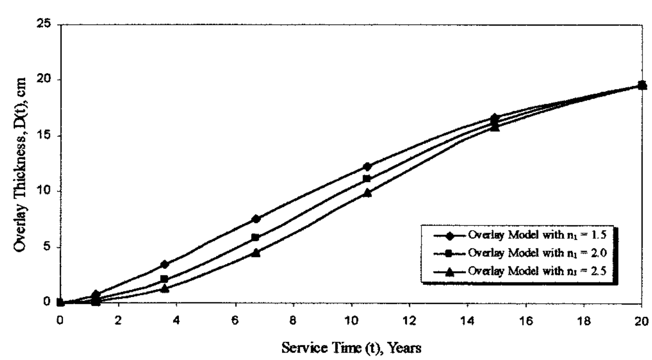

Fig. 6. Sample overlay models based on utilized performance area using American Association of State Highway and Transportation Officials design method

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Table 3. Sample Overlay Thicknesses Using Pavement Condition Indicator Model

Time (t) years	$\Delta P(t)$[a]	$F_g(t)$	Asphalt overlay thickness $[D(t)]$, cm: AASHTO method $n_2=1$	AASHTO method $n_2=1.5$[b]	AASHTO method $n_2=2$	Caltrans method $n_2=1$	Caltrans method $n_2=1.5$[b]	Caltrans method $n_2=2$
0.00	0.0	1.00	0.00	0.00	0.00	0.00	0.00	0.00
1.20	0.5	1.00	2.44	0.66	0.41	2.62	1.07	0.43
3.56	1.0	1.04	5.11	2.92	1.68	5.44	3.15	1.80
6.72	1.5	1.09	8.00	5.64	4.50	8.60	6.04	4.29
10.52	2.0	1.16	11.35	9.27	7.57	12.14	9.91	8.10
14.90	2.5	1.24	15.19	13.87	12.62	16.23	14.81	13.51
20.00	3.0	1.34	19.66	19.66	19.66	21.03	21.03	21.03

[a]The same as ΔPSI_i provided in Table 1.

[b]Selected as the best appropriate model.

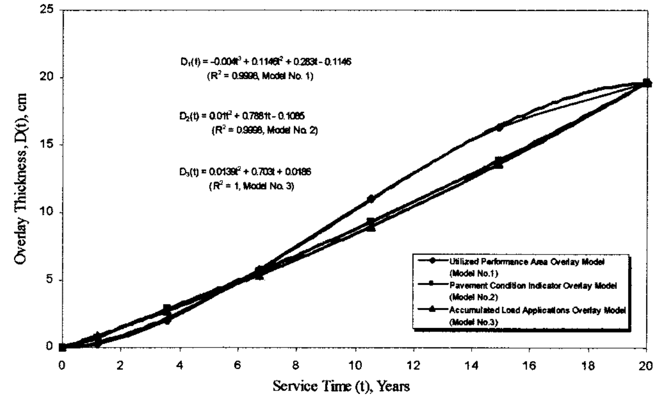

Fig. 7. Sample overlay models using American Association of State Highway and Transportation Officials design method

nesses to those used on similar pavement structures as obtained from historical records of pavement rehabilitation.

The estimated overlay thicknesses using the pavement condition indicator model are provided in Table 3 for both the AASHTO and Caltrans design methods. The difference in the initial and terminal pavement condition indicator ($\Delta P = \Delta PSI = P_0 - P_t$) is 3.0. The tabulated thicknesses are obtained using Eq. (9c) for the AASHTO method with (SN_o) replaced by (SN_1) and Eq. (12c) for the Caltrans method with (GE_o) replaced by (GE_1). The sample results are again obtained for three different model power values (n_2) with the power value of 1.5 selected as the most appropriate based on the author's best judgment as explained earlier. In Eqs. (9c) and (12c), the pavement condition indicator (P) is replaced by the PSI.

Table 4 provides similar sample results using the accumulated load applications model. These results are obtained using Eq. (9d) for the AASHTO method and Eq. (12d) for Caltrans method. The models with power value (n_3) of 1.0 seem to provide the most appropriate overlay design thicknesses. The used accumulated 80 kN ESAL applications $N(t)$ and service time (t) are obtained from the generated sample performance curve parameters provided in Table 1, namely, the last two columns of the table. The presented sample overlay thicknesses are based on an assumed annual traffic growth rate (r) of 3%.

The three selected overlay solutions, as obtained from Tables 2–4 (i.e., $n_1=2.0$, $n_2=1.5$, $n_3=1.0$), provide relatively close results as evidenced from Fig. 7. The figure shows three curves corresponding to the selected three sample overlay thickness models using the AASHTO design method. The presented sample overlay models corresponding to the pavement condition indicator and accumulated load applications models (Model Nos. 2 and 3) are almost identical as evidenced from the trend of their depicted curves. The utilized performance area model (Model No. 1) deviates from this trend at about 6 years of age when it starts providing higher overlay thickness when compared to the two other models. This is possibly the age when pavement starts showing signs of deterioration, therefore, the pavement can benefit from the additional overlay thickness provided by this model. Also, Model No. 1 provides lower overlay thickness during the first 6 years, which is the period the pavement rarely requires any rehabilitation. Therefore, Model No. 1 is considered to be superior to the two other models. Best-fit curves have been generated for the selected three overlay models with the corresponding mathematical equations provided in Fig. 7. The resulting equations are only time-dependent and polynomial in form with almost perfect coefficient of determination (R^2). Fig. 7 shows almost two identical curves for each model with one curve

Table 4. Sample Overlay Thicknesses Using Accumulated Load Applications Model

Time (t) years	$N(t)$[a] $\times 10^6$	$F_g(t)$	Asphalt overlay thickness ($D(t)$), cm: AASHTO method $n_3=0.5$	AASHTO method $n_3=1.0$[b]	AASHTO method $n_3=1.5$	Caltrans method $n_3=0.5$	Caltrans method $n_3=1.0$[b]	Caltrans method $n_3=1.5$
0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00
1.20	0.60	1.00	3.61	0.89	0.20	3.86	0.94	0.20
3.56	1.78	1.04	6.45	2.72	1.14	6.88	2.90	1.22
6.72	3.36	1.09	9.27	5.36	3.10	9.91	5.74	3.33
10.52	5.26	1.16	12.34	8.97	6.48	13.21	9.58	6.93
14.90	7.45	1.24	15.72	13.56	11.71	16.81	14.53	12.55
20.00	10.00	1.34	19.66	19.66	19.66	21.03	21.03	21.03

[a]The same as $N_{T_{i+1}}$ provided in Table 1.

[b]Selected as the best appropriate model.

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Table 5. Sample Overlay Thickness Calculations from Different Design Methods

Road number	Subgrade modulus MPa (psi)	Design equivalent single axle load ($\times 10^3$)	Existing layer thickness h_1 (cm)	h_2 (cm)	SN_o	SN_y	SN_{eff}	Overlay design thickness (cm) Proposed method	AI method	AASHTO method
1	30.8 (4,400)	460	8	30	3.18	3.52	1.64	11.63	8.88	10.91
2	75.6 (10,800)	680	9	20	2.46	2.71	1.54	6.76	6.20	6.75
3	62.0 (8,850)	240	7	20	2.25	2.51	1.48	5.52	4.66	5.94
4	95.9 (13,700)	185	7	15	1.83	2.05	1.26	4.93	5.16	5.19
5	35.7 (5,100)	700	9	30	3.22	3.56	1.64	10.24	9.01	11.08
6	33.2 (4,750)	110	7	25	2.53	2.77	1.54	7.29	6.70	7.10

Note: AASHTO=American Association of State Highway and Transportation Officials.

connecting the plotted data points and the second one representing the derived best-fit equation.

Sample Overlay Life-Cycle Cost Analysis Presentation

The utilized performance area overlay model is selected for providing sample overlay LCCA results. The formula that represents this model is given in the following equation, which is the same equation provided in Fig. 7 for Model No. 1. Three different rehabilitation schedules have been investigated using 8, 13.33, and 20 years corresponding time intervals (Δd) and 40 years life-cycle analysis period (D). The associated overlay thicknesses are 7.4, 14.6, and 19.4 cm as obtained from the following equation using the selected time intervals. The first rehabilitation schedule consists of four plain overlay cycles at an estimated cost rate of United States $\$10/\mathrm{m}^2$

$$D_1(t) = -0.004t^3 + 0.1146t^2 + 0.283t - 0.1146 \qquad (18)$$

The second rehabilitation schedule includes two rehabilitation cycles with each consisting of an overlay combined with cold milling at an estimated cost rate of United States $\$20/\mathrm{m}^2$. Typically, when cold milling is performed, the required overlay thickness is reduced based on the ratio of 1.0 cm with cold milling being equivalent to 1.5 cm of plain overlay (Caltrans 1995). Therefore, only about 10 cm of asphalt surface thickness is required. The third rehabilitation schedule consists of one rehabilitation cycle that requires, as a minimum, complete removal of the existing asphalt layer and placement of new 19.4 cm asphalt layer with an estimated cost of United States $\$40/\mathrm{m}^2$. The assumed rehabilitation cost rates are locally estimated from prevailing market prices and the actual costs of similar works.

The overlay LCCA model presented in Eq. (14) is used to determine the present cost value (P_0) associated with each rehabilitation schedule. The three present cost values associated with the three outlined pavement rehabilitation schedules are United States \$16.55, \$15.88, and $\$15.08/\mathrm{m}^2$, respectively, assuming 5% annual discount rate. The resulting three present cost values are relatively within a close range from each other with the first rehabilitation schedule being associated with the highest cost value. Of course, this cost trend would significantly change had the costs of routine maintenance and added user been considered in the LCCA. The expected outcome in such a case would support the selection of the first rehabilitation schedule (FHWA 1994; Abaza 2002).

Sample Actual Overlay Design Application

A sample of six village access roads from the district of Nablus, West Bank, are considered for overlay design using the proposed performance-based method, the AI method, and the AASHTO method. The roads were newly constructed about 10 years ago and paved using asphalt surface and aggregate base. Table 5 provides for each road the subgrade resilient modulus, projected 10 years overlay traffic, existing layers' thicknesses, and the original design SN_o used to generate the corresponding performance curve. The utilized performance area model is used to estimate the overlay thickness using a model power value of 1.5. The initial and terminal PSI values of 4.3 and 2.0 are, respectively, used to generate the relevant performance curves. The overlay thicknesses are estimated using 10 years service time ($t = 10$), and they are listed under "Proposed method" in Table 5. The relative strength coefficient is assumed to be 0.44 in determining the asphalt overlay thickness.

The AI method is then used to estimate the overlay thicknesses as indicated by Eq. (1). The new full-depth asphalt pavement is obtained from the relevant AI design chart based on the design parameters provided in Table 5 and using 15. 6°C (60°F) mean annual air temperature (AI 1991). The layer conversion factors are estimated from the relevant AI table to be 0.5 and 0.1 for asphalt surface and aggregate base, respectively. The AASHTO method as presented in Eq. (2) is used to estimate the overlay thicknesses assuming the value for the remaining life factor to be 1.0 as recommended by Elliot (1989). Because the NDT testing is not locally available for determining the existing pavement moduli, the proposed Eq. (13), with a power of 0.5, is used to estimate the total effective structural number of the existing pavement (SN_{eff}). The structural number associated with the overlay (SN_Y) is estimated based on the design 80 kN ESAL applications and subgrade modulus provided in Table 5. Table 5 shows that the presented sample overlay design thicknesses as derived from the three different methods are within close range from each other. Therefore, the proposed performance-based method is definitely a vital alternative to the practitioner.

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Conclusions and Recommendations

The presented performance-based asphalt overlay models provide a reliable alternative to the currently used overlay design models. The basic principle used in establishing the presented models is very similar to the one used by other models, which is compensating the existing pavement structure for the loss in strength it has endured over past service time. The performance curve parameters are the main input data needed to use the presented models, which can be estimated from the outlined AASHTO prediction model or from historical records of pavement distress. The performance-based overlay models can then be investigated as done in the sample presentation to yield an appropriate model based on experience and engineering judgment. The final outcome is a unique overlay design model generated for each pavement structure (project), which can be used to estimate the required overlay thickness at any given future time. Performance curve parameters can be generated, according to the AASHTO model, using an incremental change in the present serviceability index smaller than the 0.5 value used in the sample presentation. A 0.1 value can easily be deployed if a computer program is used, which would yield performance parameters with a higher degree of accuracy. The power value associated with the presented overlay models can be estimated using 0.5 increments. The appropriate power value generally falls between 1 and 3.

The overlay thickness estimated from the presented models can be applied as plain asphalt overlay up to a certain pavement age to be determined based on field assessment of pavement condition. The age associated with plain overlay typically does not exceed 10 years of service time. Overlay thicknesses associated with advanced aging times require additional corrective measures such as cold milling or complete removal of the existing asphalt surface layer. Pavement management requires undertaking corrective pavement rehabilitation measures at optimum scheduling times. The optimum pavement rehabilitation scheduling time can only be determined as part of a complete pavement life-cycle analysis (Abaza 2002). Therefore, it is recommended that such an optimum rehabilitation scheduling time be determined, which is typically located within the first half of the pavement expected service life. Plain overlay would then be adequate if the optimum scheduling time is used. It is further recommended that a highway agency, interested in using the presented overlay models, refers to its historical rehabilitation records and compares them against the results obtained from the presented overlay models to select the model that best suits the prevailing local conditions.

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