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Empirical-Markovian model for predicting the overlay design thickness for asphalt concrete pavement

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An Empirical-Markovian model has been developed to predict the overlay design thickness for asphalt concrete pavement from relevant design parameters. The Empirical-Markovian model mainly predicts the structural capacity of overlaid pavement as a function of the structural capacity associated with original pavement, annual traffic growth rate, rehabilitation scheduling time, and two calibration constants. The structural capacity is evaluated using either the structural number or gravel equivalent deployed by the American Association of State Highway and Transportation Officials and Caltrans design methods for flexible pavement, respectively. The Empirical-Markovian model provides the practitioner with two options as related to the performance of overlaid pavement. The first option enforces the performance of overlaid pavement to be similar to that of the original pavement, an objective achieved by requiring the deterioration transition probabilities for overlaid pavement to be the same as the corresponding ones for original pavement. The second option imposes improved performance of overlaid pavement compared to that of the original pavement, an objective accomplished by requiring the deterioration transition probabilities of overlaid pavement to be lower than the corresponding values associated with original pavement. The two calibration constants can be estimated by either minimising the sum of squared errors applied to historical records of pavement distress (forward approach) or a backward solution of the developed Empirical-Markovian model mainly relying on historical records of pavement rehabilitation. Two case studies are presented to demonstrate the use of both forward and backward approaches with results seem to be in line with the common practice.

Keywords: flexible pavement; Markovian processes; pavement performance; overlay design; pavement rehabilitation; pavement management

1. Introduction

Preservation of the roadway network is vital for the advancement and prosperity of any nation as it results in providing good and safe roadway operating conditions. The pavement structure is probably the most important element of any roadway and maintaining it in good condition not only reduces the overall life-cycle cost but it also positively reflects on the life standards and ethics of any nation. The pavement structure can typically be preserved through the application of routine maintenance and rehabilitation (M&R) works. However, rehabilitation works not only provide major enhancement of the pavement structure but it can substantially extend its service life (Huang, 2004; Mamlouk & Zaniewski, 1998). Pavement rehabilitation strategies typically include plain overlay, cold milling and overlay, and reconstruction (Abaza, 2002).

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The placement of an asphalt overlay is commonly used by many highway agencies to rehabilitate existing flexible and rigid pavements (Abaza & Murad, 2009; Zhou, Hu, & Scullion, 2010).

There are practically two different methods for overlay design classified as empirical and mechanistic-empirical (M-E) similar to the design of new pavement structure (Huang, 2004; Maji, Singh, & Chawla, 2016). However, the main objective of any overlay design procedure is to compensate existing pavement for the strength loss it has endured over time while accounting for the anticipated increases in traffic load applications. The most popular M-E method is the one that relies on non-destructive testing to obtain pavement surface deflections. It is deployed by most State highway agencies wherein surface deflections are measured using either the Dynaflect or Falling Weight Deflectometer (Gedafa, Hossain, Romanoschi, & Gisi, 2010; Tutumluer & Sarker, 2015a). The overlay design thickness is then estimated based on the back-calculation of the multilayer linear elastic system (Asphalt Institute [AI], 1996; Hoffman, 2003; Mallela, Titus-Glover, Singh, Darter, & Chou, 2008; Tutumluer & Sarker, 2015b). The pavement remaining strength is calculated using the modified layer coefficients (AASHTO, 1993; Tutumluer & Sarker, 2015b). However, local agencies generally lack the resources to conduct mechanical testing of pavement deflection and they mostly rely on empirical models to estimate the modified layer coefficients. Unfortunately, this approach often leads to uneconomical rehabilitation practices (Sarker, Mishra, Tutumluer, & Lackey, 2015).

Pavement performance is not only a key pavement design parameter but it is also importantly required for several applications related to pavement rehabilitation and management. Pavement performance defines the pavement service condition over time by means of a performance curve. The pavement service condition can be quantified using appropriate indicators such as the present serviceability index (PSI), pavement condition index (PCI), and international roughness index (IRI), which can be predicted using stochastic modelling. In particular, Markovian processes have been extensively used to predict pavement service condition over time for pavement management applications (Abaza, 2017; Abaza & Murad, 2009; Hong & Wang, 2003; Lethanh & Adey, 2013; Meidani & Ghanem, 2015). Pavement performance curves can be developed at the project level using Markovian processes with the deterioration transition probabilities (i.e. deterioration rates) representing the main input parameters (Abaza, 2017). Researchers have used different forms of the Markovian processes including discrete-time semi-Markov chain, exponential hidden Markov chain, Poisson hidden Markov chain, random Markov chain, and recurrent Markov chain (Abaza, 2017; Lethanh & Adey, 2013; Lethanh, Kaito, & Kobayashi, 2014; Meidani & Ghanem, 2015; Yang, Lu, Gunaratne, & Dietrich, 2006; Zhang & Gao, 2012).

Therefore, it is proposed to develop empirical-Markovian-based model to estimate the overlay design thickness under two assumption options. The first assumption option requires both overlaid and original pavements to exhibit similar deterioration trends as represented by their corresponding performance curves, an option sought when the performance of the original pavement is a good one. The second assumption option requires the deterioration rates for overlaid pavement to be lower than the corresponding ones associated with original pavement, an option sought to construct pavements with improved performance. In essence, the first option can be satisfied by requiring the deterioration transition probabilities associated with both pavements to be equal in values, while the second option requires the transition probability ratios of both original and overlaid pavements to be higher than one. The proposed overlay design approach is expected to be of a particular interest to local agencies as its main requirement is the periodical collection of pavement distress data. In addition, a simplified backward approach is presented to estimate the calibration constants associated with the proposed Empirical-Markovian model mainly replying on historical records of pavement rehabilitation.

2. Pavement performance prediction: an overview

Prediction of pavement performance has been performed using the discrete-time Markov model. Different versions of the Markov model have been used with the main difference being the form of the deployed transition probability matrix. Equation (1) presents the non-homogenous discrete-time Markov model wherein a unique transition probability matrix is used for each transition (i.e. time interval). The main function of the Markov model is to predict the future state probabilities, $\mathbf{S^{(n)}}$, at the end of an analysis period comprised of (n) transitions. This requires an estimate of the initial state probabilities, $\mathbf{S^{(0)}}$, in addition to the transition probability matrices, $\mathbf{P(k)}$. The state probabilities are normally defined in relation to the deployed pavement condition states wherein condition states are defined using key performance indicators such as the PSI and PCI. The state probabilities are typically defined as the fractions (or percentages) of pavements that belong to various condition states at any given time. For example, all new pavements (100%) normally belong to the excellent state (state **1**), whereas totally damaged pavements typically at the end of the service life belong to the worst state (state **m**). In the transitional time, pavements will transit from state **1** to other states depending on the transition probabilities until they all reach state **m** after a certain period of time in the absence of M&R works. Equation (1) indicates that the sum of the state probabilities must add up to one at any given time.

$$\mathbf{S^{(n)}} = \mathbf{S^{(0)}} \left(\prod_{\mathbf{k=1}}^{\mathbf{n}} \mathbf{P(k)} \right), \tag{1}$$

where

$$S^{(n)} = (S_1^{(n)}, \mathrm{S}_2^{(n)}, \mathrm{S}_3^{(n)}, \ldots, \mathrm{S}_m^{(n)}),$$

$$S^{(0)} = (S_1^{(0)}, \mathrm{S}_2^{(0)}, \mathrm{S}_3^{(0)}, \ldots, \mathrm{S}_m^{(0)}),$$

$$\sum_{i=1}^{m} S_i^{(k)} = 1.0.$$

The transition probability matrix is ($m \times m$) square matrix with (m) being the number of deployed pavement condition states. The transition matrix generally contains the transition probabilities ($P_{i,j}$), which indicate the probability of transiting from condition state (**i**) to state (**j**) after the elapse of one transition. The entries above the main diagonal represent the deterioration transition probabilities ($P(k)_{i,j}$; $i < j$), entries below the main diagonal indicate the improvement transition probabilities ($P(k)_{i,j}$; $i > j$), and entries along the main diagonal denote the probabilities of remaining in the same condition state ($P(k)_{i,j}$; $i = j$) for the kth transition. Abaza (2017) used a simplified form of the transition probability matrix as defined in Equation (2) wherein the improvement transition probabilities are assigned zero values in the absence of M&R works. Therefore, Equation (2) can be used to predict the performance of an original pavement structure in the absence of M&R works.

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

where

$$P(k)_{i,i} + P(k)_{i,i+1} = 1.0, \quad P(k)_{m,m} = 1.0,$$
$$0 \le P(k)_{i,i} \le 1.0, \quad 0 \le P(k)_{i,i+1} \le 1.0.$$

In addition, the transition matrix indicated by Equation (2) assumes that pavement deterioration can take place in one step, thus, requiring the use of only one set of deterioration transition probabilities, namely $P(k)_{i,i+1}$. The validity of this assumption mainly depends on the transition matrix size and transition length. It is more valid as the matrix size (m) gets larger and the transition length becomes smaller. Abaza (2017) reported that (10×10) transition matrix and 1-year transition length are sufficient conditions to satisfy this assumption. It is to be also noted that the sum of any row in the transition matrix must add up to one.

Pavement performance is typically defined over the deployed analysis period using an appropriate pavement condition indicator such the PCI, distress rating (DR), or PSI. The future pavement performance can be predicted based on the state probabilities estimated using Equation (1). The pavement distress ratings at the project level, DR(k), can be predicted using Equation (3) as a function of the state mean DRs (B_i) and state probabilities, $S_i^{(k)}$, associated with the kth transition. The state mean DRs are defined using 10-point DR ranges considering a Markov model with 10 condition states. A scale of 100 points has been used for the DR with higher ratings indicating better pavement. The required state probabilities can be estimated from Equation (1) provided that the relevant transition probability matrices are available over an analysis period of (n) transitions.

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

where

$$S^{(k)} = \begin{cases} S_1^{(k)}, & 90 < \mathrm{DR} \le 100, \quad B_1 = 95 \\ S_2^{(k)}, & 80 < \mathrm{DR} \le 90, \quad B_2 = 85 \\ S_3^{(k)}, & 70 < \mathrm{DR} \le 80, \quad B_3 = 75 \\ \vdots & \vdots \qquad\qquad\qquad \vdots \\ S_{10}^{(k)}, & 0 \le \mathrm{DR} \le 10, \quad B_{10} = 5. \end{cases}$$

Once the distress ratings, DR(k), are predicted over an analysis period comprised of (n) transitions, then the corresponding performance curve can be developed for a specific pavement project as shown in Figure 1. Two distinct performance trends can be identified from Figure 1. The first one indicates superior performance as it is associated with increasingly higher deterioration transition probabilities, while the second one shows inferior performance as it is associated with decreasingly lower deterioration transition probabilities (Abaza, 2017). Abaza (2016) proposed simplified models to estimate the actual (observed) DR mainly based on cracking and deformation. However, the DR can be replaced by more popular indicators such as PSI and PCI to be estimated using relevant published procedures and models.

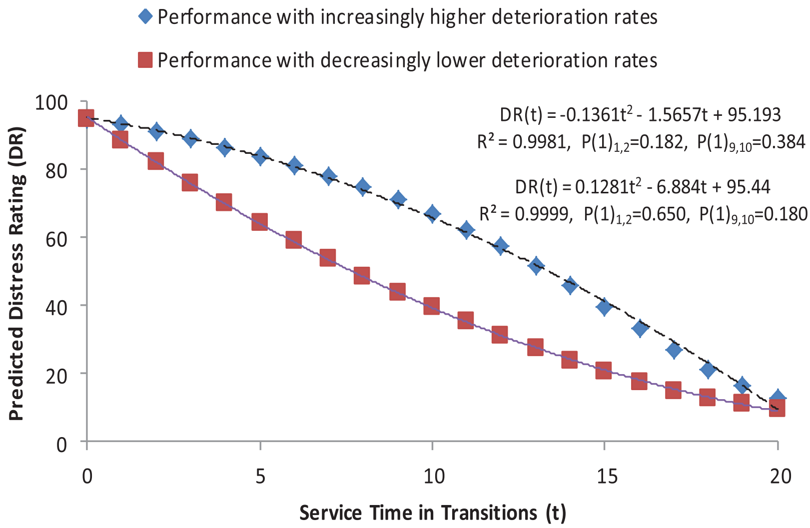

Figure 1. Sample pavement performance curves for original pavement predicted using non-homogenous Markov chain.

3. Methodology

The main drawback of using the non-homogenous Markov model presented in Equation (1) is the need to estimate (n) transition probability matrices for an analysis period comprised of (n) transitions. This is because adequate pavement distress records may not be available over the entire analysis period. Therefore, Abaza (2017) proposed an empirical model to predict the future non-homogenous deterioration transition probabilities based on the corresponding present values as defined in Equation (4). A traffic load factor raised to power (A) is used to account for the increasingly higher load applications, while a pavement strength factor raised to power (B) is introduced to capture the impact of decreasingly lower pavement strength over time. The impacts of both factors, in the absence of any M&R works, will result in higher future deterioration transition probabilities as traffic loading will increase and pavement strength will decrease over time. The model exponents (A and B) can be estimated from the calibration procedure which relies on the minimisation of the sum of squared errors (SSE) as outlined in Abaza (2017).

$$P(k+1)_{i,i+1} = P(k)_{i,i+1}\left(\frac{\Delta W(k+1)}{\Delta W(k)}\right)^{A}\left(\frac{\mathrm{SC}(k-1)}{\mathrm{SC}(k)}\right)^{B} \quad (k = 1, 2, 3, \ldots, n). \tag{4}$$

An empirical model similar to the one presented in Equation (4) is proposed to predict the future non-homogenous deterioration transition probabilities, $P(t+k)_{i,i+1}$, associated with overlaid pavement based on the corresponding values, $P(k)_{i,i+1}$, associated with the original pavement as indicated by Equation (5). The main factors used in Equation (5) are the traffic load and pavement strength factors raised to the powers (A and B), respectively. The load factor is a ratio between the 80 kN single-axle load applications (ESAL) expected to travel the overlaid pavement during the $(t+k)$th transition, $\Delta W(t+k)$, to the corresponding value, $\Delta W(k)$, associated with the original pavement during the kth transition. Similarly, the strength factor is a ratio between the structural capacity associated with the original pavement at the $(k-1)$th transition, $\mathrm{SC}(k-1)$, to the corresponding value associated with the overlaid pavement, $\mathrm{SC}(t+k-1)$, at the $(t+k-1)$th transition as shown in Figure 2. Both ratios are expected to be greater than one as load applications will increase and pavement strength will decrease, thus, resulting in

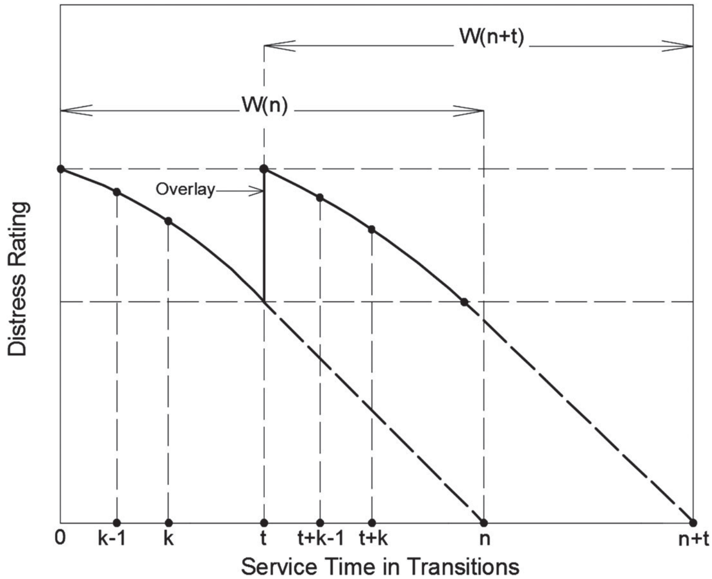

Figure 2. Typical overlay scheduling plan with similar performance curves for both overlaid and original pavements.

higher deterioration transition probabilities over time. Figure 2 shows (t) to be the rehabilitation scheduling time in transitions, which is the same as the age associated with original pavement.

$$P(t+k)_{i,i+1} = P(k)_{i,i+1}\left(\frac{\Delta W(t+k)}{\Delta W(k)}\right)^{A}\left(\frac{\mathrm{SC}(k-1)}{\mathrm{SC}(t+k-1)}\right)^{B} \quad (k = 1, 2, \ldots, t). \tag{5}$$

However, the main objective from proposing Equation (5) is not to predict the deterioration transition probabilities but to derive a model that can be used to estimate the required strength for overlaid pavement. Therefore, Equation (5) is solved to obtain an expression for the structural capacity associated with overlaid pavement, SC($t+k-1$), in terms of the other remaining parameters. The outcome is Equation (6) which indicates that the required structural capacity is a function of the structural capacity associated with original pavement, deterioration transition probabilities associated with both overlaid and original pavements, traffic load factor, and two calibration constants (A and B).

$$\mathrm{SC}(t+k-1) = S(k-1)\left(\frac{P(k)_{i,i+1}}{P(t+k)_{i,i+1}}\right)^{1/B}\left(\frac{\Delta W(t+k)}{\Delta W(k)}\right)^{A/B} \quad (k = 1, 2, \ldots, t). \tag{6}$$

There are two practical cases of pavement deterioration rates (i.e. transition probabilities) that can be considered in the process of estimating the structural capacity for overlaid pavement as described next.

3.1. *Case I: equal pavement deterioration rates*

A special case of Equation (6) can be derived to estimate the required structural capacity of overlaid pavement under the assumption that both overlaid and original pavements are associated with similar performances. This assumption can be achieved by requiring the two sets of deterioration transition probabilities [$P(t+k)_{i,i+1}$ and $P(k)_{i,i+1}$] associated with both overlaid and original pavements, respectively, to be equal in values. This assumption implies that the performance curve associated with overlaid pavement will resemble that of the original pavement as shown in Figure 2 with the same service life of (n) transitions. This assumption is a practical one as long as the performance of original pavement is a desirable one.

It is therefore required to set the deterioration transition probabilities, $P(t+k)_{i,i+1}$, associated with overlaid pavement to be equal to the corresponding values, $P(k)_{i,i+1}$, associated with original pavement. The outcome of this requirement results in the derivation of Equation (7) as obtained from the simplification of Equation (6). This will ensure the performance curves associated with both pavements to be similar as shown in Figure 2. According to Equation (7), the structural capacity required for overlaid pavement at the $(t+k-1)$th transition is a function of the structural capacity associated with original pavement at the $(k-1)$th transition and the corresponding traffic load factor raised to the power (A/B). The model exponents (A and B) are assumed to be the same as the ones used in Equation (4), which is the model applicable to original pavement. This is a reasonable assumption since both pavements are expected to exhibit similar performance trends as depicted in Figure 2.

$$\mathrm{SC}(t+k-1) = \mathrm{SC}(k-1)\left(\frac{\Delta W(t+k)}{\Delta W(k)}\right)^{A/B} \quad (k = 1, 2, \ldots, t), \tag{7}$$

where

$$\Delta W(t+k) = W(t+k) - W(t+k-1),$$
$$\Delta W(k) = W(k) - W(k-1).$$

The accumulated traffic load applications at the kth transition, $W(k)$, in terms of the ESAL, can be obtained from multiplying the first-year load applications (W_{f}) by the corresponding traffic growth factor, GF(k), as defined in Equation (8). The deployed formula for estimating the GF(k) is the one proposed by the AI and it is a function of the uniform annual traffic growth rate (r) (AI, 1999).

$$W(k) = W_{\mathrm{f}} \times \mathrm{GF}(k) = W_{\mathrm{f}}\left(\frac{(1+r)^k - 1}{r}\right). \tag{8}$$

The ratio associated with the traffic load factor can then be simplified as presented in Equation (9). The outcome of this simplification indicates that this ratio is only dependent on the traffic growth rate (r) and rehabilitation scheduling time (t) in transitions. Each transition is typically assumed to be equal to one year.

$$\frac{\Delta W(t+k)}{\Delta W(k)} = \frac{W_{\mathrm{f}} \times \mathrm{GF}(t+k) - W_{\mathrm{f}} \times \mathrm{GF}(t+k-1)}{W_{\mathrm{f}} \times \mathrm{GF}(k) - W_{\mathrm{f}} \times \mathrm{GF}(k-1)} = (1+r)^t. \tag{9}$$

Alternatively, the ratio associated with the traffic load factor can be estimated from the ratio of the accumulated load applications expected to travel the overlaid pavement during its service life, $W(n+t)$, to the design accumulated load applications associated with original pavement, $W(n)$, as shown in Figure 2. Equation (10) indicates that this load ratio results in the same term as presented in Equation (9). This implicitly states that the overlay design thickness is directly dependent on the ratio of the accumulated load applications associated with both overlaid and original pavements, which seems to be an appropriate conclusion.

$$\frac{W(n+t)}{W(n)} = \frac{W_{\mathrm{f}} \times \mathrm{GF}(n+t) - W_{\mathrm{f}} \times \mathrm{GF}(t)}{W_{\mathrm{f}} \times \mathrm{GF}(n)} = (1+r)^{t}. \tag{10}$$

The derived term for the ratio associated with the traffic load factor is then substituted in Equation (7) to yield Equation (11). However, Equation (11) is derived at the 1st transition ($k = 1$) implying that the required structural capacity for estimating the overlay design thickness at time (t) is dependent on the initial structural capacity associated with original pavement, SC(0). This is a reasonable requirement since the strength for pavement design is typically estimated at the beginning of the pavement service life.

$$\mathrm{SC}(t) = \mathrm{SC}(0)(1+r)^{tA/B}. \tag{11}$$

The structural capacity required for pavement design has typically been represented by relative strength indicators when considering empirical-based design methods. A very popular indicator is the structural number (SN) used by AASHTO in its guide for pavement design (AASHTO, 1993). Another popular relative strength indicator is the gravel equivalent (GE) used by Caltrans design manual (Caltrans, 2008). Therefore, it is proposed to replace the structural capacity used in Equation (11) by the SN and GE to yield Equation (12). However, the SN and GE associated with the asphalt concrete layer (SN_1 and GE_1) are used in Equations (12a) and (12b), respectively. This is a reasonable assumption to make as the asphalt concrete layer is the main layer that endures strength loss over time. The remaining underlying pavement layers typically experience very little strength loss especially when they are made of granular materials (Abaza, 2017).

$$\mathrm{SN}_1(t) = \mathrm{SN}_1(0)(1+r)^{tA/B}, \tag{12a}$$

$$\mathrm{GE}_1(t) = \mathrm{GE}_1(0)(1+r)^{tA/B}. \tag{12b}$$

According to Equation (12), the design structural capacity, $\mathrm{SN}_1(t)$ or $\mathrm{GE}_1(t)$, for the asphalt concrete layer associated with overlaid pavement is to be estimated based on the corresponding value associated with original pavement, $\mathrm{SN}_1(0)$ or $\mathrm{GE}_1(0)$, annual traffic growth rate (r), rehabilitation scheduling time (t), and two calibration exponents (A and B).

3.2. *Case II: improved pavement deterioration rates*

Occasionally, it is required to design overlaid pavement to perform better than the original pavement. This requires specifying improved deterioration transition probabilities for overlaid pavement, $P(t+1)_{i,i+1}$, compared to the corresponding values associated with original pavement, $P(1)_{i,i+1}$, which can be done through the application of Equation (6). The resulting model is as indicated by Equation (13) which is equivalent to Equation (12) but with the inclusion of the deterioration transition probabilities associated with the 1st transition ($k = 1$). For a Markovian chain with (m) condition states, there are ($m - 1$) deterioration transition probabilities which can result in ($m - 1$) different values of the required structural capacity, $\mathrm{SC}_1(t) = \mathrm{SN}_1(t)$ or $\mathrm{GE}_1(t)$, associated with asphalt concrete layer according to the following equation:

$$\mathrm{SC}_1(t) = \mathrm{SC}_1(0)(1+r)^{tA/B}\left(\frac{P(1)_{i,i+1}}{P(t+1)_{i,i+1}}\right)^{1/B} \quad (i = 1, 2, \ldots, m-1). \tag{13}$$

However, for improved pavement performance, the deterioration transition probabilities associated overlaid pavement need to be lower in values compared to the corresponding ones associated with original pavement. This means the transition probability ratio ($\mathrm{TP_R}$) deployed in Equation (13) has to be greater than one. Therefore, one simple application of Equation (13) is to assume a single desirable $\mathrm{TP_R}$ to be greater than one as indicated by Equation (14), which avoids explicitly using the relevant deterioration transition probabilities. Equation (12) is a special case of Equation (14) wherein the $\mathrm{TP_R}$ parameter is assigned the value of one.

$$\mathrm{SC}_1(t) = \mathrm{SC}_1(0)(1+r)^{tA/B}(\mathrm{TP_R})^{1/B} \quad (\mathrm{TP_R} > 1.0), \tag{14}$$

where

$$\mathrm{TP_R} = \frac{P(1)_{i,i+1}}{P(t+1)_{i,i+1}}.$$

Alternatively, the initial and terminal deterioration transition probabilities, $P(1)_{1,2}$ and $P(1)_{m-1,m}$, associated with the 1st transition can be used to provide two different estimates of the structural capacity associated with overlaid pavement as provided in Equations (15a) and (15b), respectively. The larger of the two $\mathrm{SC}_1(t)$ values can then be used in the estimation of the required overlay design thickness. Abaza and Murad (2009) proposed a simplified approach to predict pavement performance solely based on the initial and terminal deterioration transition probabilities with the remaining deterioration transition probabilities estimated using linear interpolation. These two transition probability values provide the main input data needed to generate the performance curve for a particular pavement project. They also greatly influence the shape of the performance curve expected to define pavement deterioration over time. This approach was used in developing the two sample performance curves shown in Figure 1.

$$\mathrm{SC}_1(t) = \mathrm{SC}_1(0)(1+r)^{tA/B}\left(\frac{P(1)_{1,2}}{P(t+1)_{1,2}}\right)^{1/B}, \tag{15a}$$

$$\mathrm{SC}_1(t) = \mathrm{SC}_1(0)(1+r)^{tA/B}\left(\frac{P(1)_{m-1,m}}{P(t+1)_{m-1,m}}\right)^{1/B}. \tag{15b}$$

Therefore, Equations (15a) and (15b) can be used to obtain two estimates of the structural capacity of the asphalt concrete layer associated with overlaid pavement for improved pavement performance by simply specifying initial and terminal deterioration transition probabilities smaller than the corresponding values associated with original pavement. The larger of the two estimates should be selected for overlay design.

3.3. *Estimation of overlay design thickness*

The overlay structural capacity required at rehabilitation scheduling time (t), $\mathrm{SC}_0(t)$, is typically estimated using Equation (16). It is defined as the difference between the structural capacity associated with overlaid pavement, $\mathrm{SC}(t)$, and the existing pavement structural capacity, $\mathrm{SC}_{\mathrm{eff}}(t)$ (Huang 2004; Nam, An, Kim, Murphy, & Zhang, 2016, Tutumluer & Sarker, 2015a, 2015b). The existing (effective) structural capacity for a pavement structure can be estimated from multiplying the effective layer coefficients, $a_j''(t)$, or effective layer gravel factors, $\mathrm{Gf}_j''(t)$, by the corresponding layer thicknesses (D_j) according to AASHTO and Caltrans, respectively, as indicated by the following equation (AASHTO, 1993; Caltrans, 2008):

$$\mathrm{SC}_0(t) = \mathrm{SC}(t) - \mathrm{SC}_{\mathrm{eff}}(t), \tag{16}$$

where

$$\mathrm{SC}_{\mathrm{eff}}(t) = \mathrm{SN}_{\mathrm{eff}}(t) = \sum_j a''_j(t) \times D_j,$$

$$\mathrm{SC}_{\mathrm{eff}}(t) = \mathrm{GE}_{\mathrm{eff}}(t) = \sum_j \mathrm{Gf}''_j(t) \times D_j.$$

Alternatively, the required overlay structural capacity can be estimated as the difference between the asphaltic structural capacity, $\mathrm{SC}_1(t)$, required at rehabilitation time (t) and the corresponding effective value, $\mathrm{SC}_{1\mathrm{eff}}(t)$, as defined in Equation (17). This is consistent with the assumption that underlying pavement layers experience minor strength losses over time. The asphaltic structural capacity required at rehabilitation time (t), $\mathrm{SC}_1(t)$, is to be estimated from the Empirical-Markovian models presented earlier, namely Equations (12) and (14).

$$\mathrm{SC}_0(t) = \mathrm{SC}_1(t) - \mathrm{SC}_{1\mathrm{eff}}(t). \tag{17}$$

The existing (effective) asphaltic structural capacity, $\mathrm{SC}_{1\mathrm{eff}}(t)$, is typically estimated using the corresponding existing layer coefficient, $a''_1(t)$, or existing gravel factor, $\mathrm{Gf}''_1(t)$, according to AASHTO and Caltrans, respectively. They are estimated from non-destructive testing typically performed to measure pavement surface deflections which are then used to back-calculate the pavement layer moduli (Hong, 2014; Huang 2004; Jimoh, Itiola, & Afolabi, 2015). In this paper, it is proposed to use a simplified approach to estimate the existing asphaltic structural capacity by applying a remaining strength factor, $F(t)$, as defined in Equation (18). The remaining strength factor, $F(t)$, is therefore introduced in Equations (18a) and (18b) to allow for reducing the structural capacity associated with the existing asphalt concrete layer according to AASHTO and Caltrans design methods, respectively.

$$\mathrm{SN}_0(t) = \mathrm{SN}_1(t) - F(t)\mathrm{SN}_1(0), \tag{18a}$$

$$\mathrm{GE}_0(t) = \mathrm{GE}_1(t) - F(t)\mathrm{GE}_1(0). \tag{18b}$$

The required overlay design thickness can then be estimated using Equation (19). The overlay structural number, $\mathrm{SN}_0(t)$, is divided by the asphalt layer coefficient ($a_0 = 0.44$) to yield the overlay design thickness, $h_0(t)$, in centimetres according to AASHTO. Similarly, the overlay gravel equivalent, $\mathrm{GE}_0(t)$, is divided by the asphalt gravel factor ($\mathrm{Gf}_0 = 1.9$) to obtain the overlay thickness in centimetres according to Caltrans. The originally calculated layer thicknesses are in inches and feet according to AASHTO and Caltrans, respectively, therefore conversion factors to centimetres are introduced in the following equations:

$$h_0(t) = 2.5 \times \left(\frac{\mathrm{SN}_0(t)}{a_0}\right), \tag{19a}$$

$$h_0(t) = 30 \times \left(\frac{\mathrm{GE}_0(t)}{\mathrm{Gf}_0}\right). \tag{19b}$$

The remaining strength factor, $F(t)$, can be estimated from both the destructive and non-destructive testing of pavement (Hong 2014; Huang 2004). Abaza (2017) proposed to define the remaining strength as the ratio of the relative strength coefficient for the asphalt concrete layer at the time of overlay to the corresponding design value associated with the original pavement. In another study, Abaza and Murad (2009) proposed to use the ratio of the area falling under the performance curve between overlay time and end of service life to the total area under curve. In this study, it is proposed to use the ratio of the pavement distress rating, DR(t), at the time of overlay to the initial DR value, DR(0), as presented in the following equation:

$$F(t) = \frac{\mathrm{DR}(t)}{\mathrm{DR}(0)}. \tag{20}$$

The required DR(t) can either be the predicted value as obtained from Equation (3) or the observed value at the time of rehabilitation. Generally, there are two types of pavement performance as depicted in Figure 1 (Abaza and Murad 2009). The first type is a superior performance associated with increasingly higher deterioration transition while the second one is an inferior performance associated with decreasingly lower deterioration transition probabilities. It is expected that the remaining strength factor, $F(t)$, as estimated from Equation (20) will provide a reasonable estimate of the pavement remaining strength associated with the asphalt concrete layer. This is especially true when the relevant distress assessment is conducted to mainly account for the structural deficiencies associated with the asphalt concrete layer, namely those related to cracking and deformation (Abaza, 2017; Hong, 2014).

3.4. *Back-calculation of model calibration constants*

The two calibration constants (A and B) associated with the presented Empirical-Markovian models can be estimated using different techniques. As outlined earlier, Abaza (2017) applied a forward approach mainly replying on the minimisation of SSE to estimate the two model exponents (A and B) wherein the errors defined as the differences between the predicted and observed annual DRs. This approach can be applied at the project level provided adequate records of annual DRs are available (i.e. observed ratings) while the corresponding predicted values are estimated from stochastic modelling as defined in Equation (3). This forward approach requires the state probabilities and non-homogeneous transition probabilities to be estimated for a specified analysis period (Abaza, 2017).

A new technique is proposed in this paper which is referred to as a backward approach since it attempts to estimate the two calibration constants from historical rehabilitation records available at the network level. The required rehabilitation records are readily available to highway agencies which mainly include the rehabilitation time and overlay thickness for a roadway sample with similar loading conditions and materials characteristics. Equation (21) is mainly a reproduction of Equation (12) proposed to estimate the asphaltic structural capacity of the jth roadway at rehabilitation time (t) from related variables with the ratio (A/B) is replaced by the calibration constant $C(j)$.

$$\mathrm{SC}_1(t,j) = \mathrm{SC}_1(0,j)(1+r)^{t(j)C(j)}. \tag{21}$$

Equation (21) is simply solved for the constant $C(j)$ as indicated by Equation (22) with the other parameters are assumed to be known for the jth roadway (project). In addition to the rehabilitation time (t), it is required to know the original asphaltic structural capacity, $\mathrm{SC}_1(0,j)$, overlay structural capacity, $\mathrm{SC}''_0(t,j)$, as actually constructed, and the remaining strength factor, $F(t)$, at the time of rehabilitation. The total asphaltic structural capacity associated with overlaid pavement at the time of rehabilitation, $\mathrm{SC}''_1(t,j)$, is essentially the sum of the overlay structural capacity and remaining asphaltic structural capacity. An average (C) value can be computed from the derived $C(j)$ values to provide a reasonable representation of the project sample being considered assuming Empirical-Markovian model with equal pavement deterioration rates.

$$C(j) = \frac{\log \mathrm{SC}''_1(t,j) - \log \mathrm{SC}_1(0,j)}{t(j)\log(1+r)}, \tag{22}$$

where

$$\mathrm{SC}''_1(t,j) = \mathrm{SC}''_0(t,j) + F(j) \times \mathrm{SC}_1(0,j).$$

Similarly, the Empirical-Markovian model for improved pavement deterioration rates as presented in Equation (14) is reproduced in Equation (23) with the exponents (A/B) and ($1/B$) are replaced by the constants $C(j)$ and $D(j)$, respectively. As an approximation, the constants $C(j)$ are assumed to have the same values as obtained from Equation (22). Equation (23) can now be solved for the constants $D(j)$ as presented in Equation (24). The required asphaltic structural capacities (SC) can either be replaced by the corresponding SNs or GEs.

$$\mathrm{SC}_1(t,j) = \mathrm{SC}_1(0,j)(1+r)^{t(j)C(j)}(\mathrm{TP_R})^{D(j)}, \tag{23}$$

$$D(j) = \frac{1}{\log \mathrm{TP_R}}[(\log \mathrm{SC}''_1(t,j) - \log \mathrm{SC}_1(0,j)) - (t(j)C(j)\log(1+r))], \quad \mathrm{TP_R} > 1.0. \tag{24}$$

The different parameters used in Equation (24) are the same ones deployed in Equation (22) with the $\mathrm{TP_R}$ to be selected greater than one. Similarly, an average (D) value can be computed to represent the deterioration mechanism of the roadway sample under consideration.

4. Sample presentation

In this section, two sample problems are presented. The first one applies the proposed Empirical-Markovian model to estimate the overlay design thickness for a major urban arterial assuming equal deterioration rates. The second one attempts to use the backward solution to estimate the two calibration constants for a sample of local roads with known rehabilitation records.

4.1. *Sample problem I: major urban arterial*

The proposed Empirical-Markovian model has been used to estimate the overlay design thickness for four-lane major urban arterial located in the city of Nablus, West Bank, Palestine. The arterial is paved with flexible pavement composed of 12-cm hot-mix asphalt (HMA) surface on top of 50-cm aggregate base. The original asphaltic GE and SN are $\mathrm{GE}_1(0) = 0.7$ ft and $\mathrm{SN}_1(0) = 2.1$, respectively. This pavement structure was designed to support 5-million design ESAL, $W(n)$, over an analysis period (n) composed of 20 years (i.e. 20 transitions) with 4% annual traffic growth rate (r). The gravel factor and layer coefficient for HMA overlay are assumed to be $\mathrm{Gf}_0 = 1.9$ and $a_0 = 0.44$, respectively.

Abaza (2017) predicted the performance of this arterial using the initial and terminal deterioration transition probabilities as obtained from the empirical model presented in Equation (4). Two types of pavement performance were identified for this arterial as outlined earlier with the corresponding initial and terminal deterioration transition probabilities [$P(1)_{1,2}$ and $P(1)_{9,10}$] provided in Figure 1 for a Markov chain with 10 condition states ($m = 10$). The superior performance [i.e. $P(1)_{1,2} < P(1)_{9,10}$] had prevailed over the vast majority of the arterial pavement that was built on subgrade with good bearing capacity, while inferior performance [i.e. $P(1)_{1,2} > P(1)_{9,10}$] was spotted over few pavement sections that were constructed on poor subgrade. The corresponding performance curves (models) shown in Figure 1 can be consulted to estimate the distress rating, DR(t), as a function of service time in transitions (t) with one transition being equivalent to one year.

Equation (20) has been used to estimate the remaining strength factor, $F(t)$, with the required distress ratings, DR(t), are estimated from Figure 1. Tables 1 and 2 provide the corresponding $F(t)$ values as a function of the rehabilitation scheduling time (t) for superior and inferior pavement performances, respectively. The tables also provide the accumulated load applications, $W(t+n)$, that are expected to travel over the overlaid pavement considering a service life (n) of 20 transitions with values increasing as the rehabilitation time (t) increases. The rehabilitation time has been varied from 5 to 10 years, which is the practical time range for pavement resurfacing. Abaza (2002) proposed an optimum life-cycle analysis model and reported that the optimum overlay scheduling time is about 7–8 years. Tables 1 and 2 indicate that as the pavement service time (t) increases from 5 to 10 years, the remaining strength factor, $F(t)$, decreases from 0.882 to 0.693 in the case of superior performance and from 0.673 to 0.413 in the case of inferior performance. The asphaltic SC for overlaid pavement according to AASHTO and Caltrans, $SN_1(t)$ and $GE_1(t)$, are estimated using Equations (12a) and (12b), respectively, assuming equal deterioration rates. The model exponents (A and B) have been assigned the values of (1.4 and 1.2) for superior performance and (0.7 and 0.4) for inferior performance as reported by Abaza (2017) based on the calibration of the empirical model presented in Equation (4).

The overlay structural capacities, $SC_0(t)$, are computed using Equation (18) and converted to overlay design thicknesses using Equation (19). Table 1 indicates that the ranges for overlay design thickness are 4.15–9.81 cm and 4.48–10.59 cm in the case of superior performance, whereas Table 2 denotes the ranges for inferior performance to be 8.15–17.40 cm and 8.70–18.78 cm using Caltrans and AASHTO methods, respectively. Tables 1 and 2 also indicate an inverse relationship between the overlay design thickness and remaining strength factor as would be expected for the same roadway. Figures 3 and 4 have yielded perfect 2nd degree polynomial models ($R^2 = 1$) relating overlay design thickness to rehabilitation scheduling time. They also indicate that the Caltrans method has provided overlay design thicknesses that are about 7.4% lower than the corresponding values obtained from the AASHTO method. It can be noted that the overlay design thicknesses seem to be appropriate in the case of superior performance, while they are about 90% higher in the case of inferior performance compared to superior

Table 1. Sample overlay design thickness for superior pavement performance assuming equal deterioration rates ($A = 1.4$, $B = 1.2$).

t (yrs.)	$W(n+t) \times 10^6$	DR(t)	$F(t)$	$SC_1(t)$	$SC_0(t)$	$h_0(t)$, cm
5	6.08	83.96	0.882	0.880	0.263	4.15[a]
				2.640	0.788	4.48[b]
6	6.33	80.90	0.850	0.921	0.326	5.15
				2.764	0.979	5.56
7	6.58	77.56	0.815	0.964	0.394	6.22
				2.893	1.182	6.72
8	6.84	73.96	0.777	1.010	0.466	7.36
				3.029	1.397	7.94
9	7.12	70.08	0.736	1.057	0.542	8.56
				3.170	1.624	9.23
10	7.40	65.93	0.693	1.106	0.621	9.81
				3.319	1.864	10.59

[a]Estimated based on Caltrans method ($SC_1(0) = GE_1(0) = 0.70$ ft, $GF_0 = 1.9$).
[b]Estimated based on AASHTO method ($SC_1(0) = SN_1(0) = 2.1$, $a_0 = 0.44$).

Table 2. Sample overlay design thickness for inferior pavement performance assuming equal deterioration rates ($A = 0.7$, $B = 0.4$).

t (yrs.)	$W(n + t) \times 10^6$	DR(t)	$F(t)$	$SC_1(t)$	$SC_0(t)$	$h_0(t)$, cm
5	6.08	64.22	0.673	0.987	0.516	8.15[a]
				2.960	1.547	8.79[b]
6	6.33	58.75	0.616	1.057	0.626	9.88
				3.170	1.876	10.66
7	6.58	53.53	0.561	1.132	0.739	11.67
				3.395	2.217	12.60
8	6.84	48.57	0.509	1.212	0.856	13.52
				3.636	2.567	14.58
9	7.12	43.86	0.460	1.298	0.976	15.41
				3.895	2.929	16.64
10	7.40	39.41	0.413	1.391	1.102	17.40
				4.172	3.305	18.78

[a]Estimated based on Caltrans method ($SC_1(0) = GE_1(0) = 0.70$ ft, $GF_0 = 1.9$).
[b]Estimated based on AASHTO method ($SC_1(0) = SN_1(0) = 2.1$, $a_0 = 0.44$).

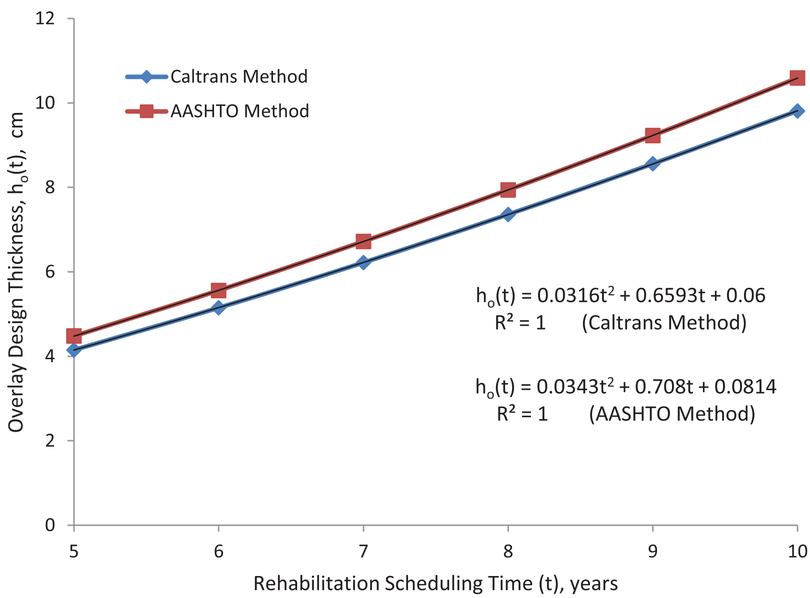

Figure 3. Sample overlay design thickness calculated using Caltrans and AASHTO methods for superior performance with equal deterioration rates ($A = 1.4$, $B = 1.2$).

performance. As mentioned earlier, in this sample pavement project only few sections were identified to exhibit inferior performance due to poor subgrade support. Therefore, reconstruction of these sections is recommended with the pavement design to be performed using the relevant subgrade bearing capacity.

4.2. *Sample problem II: local roadway sample*

A sample of 10 village access roads has been selected for the purpose of applying the backward approach to estimate the two calibration constants $C(j)$ and $D(j)$ defined in Equations (22) and

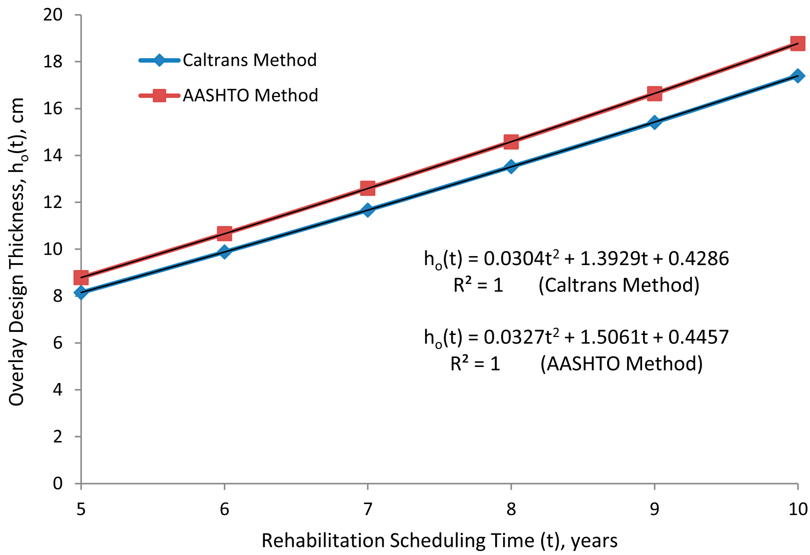

Figure 4. Sample overlay design thickness calculated using Caltrans and AASHTO methods for inferior performance with equal deterioration rates ($A = 0.7$, $B = 0.4$).

(24), respectively. These two-lane local roads connect several villages in the Nablus District with the nearby main highways and they are generally classified as low volume roads. They were reconstructed about 10 years ago ($t = 10$ years), but they are now considered for rehabilitation. Table 3 provides the average distress rating, DR(t), for each roadway as recently estimated from

Table 3. Sample overlay design thickness calculated using Caltrans method assuming superior pavement performance (DR(0) = 95, $A = 1.4$, $B = 1.2$, $t = 10$ years).

Road j	DR(t)	$F(t)$	$W(n) \times 10^3$	TI	$GE_1(0)$	$GE_1(t)$	$GE_0(t)$	$h_0(t)$, cm
1	73.6	0.775	470	8.23	0.527	0.833	0.425	6.71[a]
						0.969	0.561	8.86[b]
2	67.9	0.715	320	7.86	0.503	0.795	0.435	6.87
						0.925	0.565	8.92
3	62.3	0.656	520	8.33	0.533	0.842	0.492	7.77
						0.980	0.630	9.95
4	61.2	0.644	180	7.34	0.470	0.743	0.440	6.95
						0.865	0.562	8.87
5	59.2	0.623	350	7.94	0.508	0.803	0.486	7.67
						0.935	0.619	9.77
6	54.2	0.571	650	8.55	0.547	0.864	0.552	8.72
						1.006	0.694	10.96
7	53.4	0.562	250	7.63	0.488	0.771	0.497	7.85
						0.897	0.623	9.84
8	48.0	0.505	700	8.63	0.552	0.872	0.593	9.36
						1.015	0.736	11.62
9	46.3	0.487	580	8.44	0.540	0.854	0.591	9.33
						0.994	0.731	11.54
10	41.9	0.441	620	8.50	0.544	0.860	0.620	9.79
						1.001	0.761	12.02

[a]Estimated assuming equal deterioration rates ($TP_R = 1.0$).
[b]Estimated assuming improved deterioration rates ($TP_R = 1.2$).

prevailing pavement defects. The table also provides the original design ESAL, $W(n)$, which has been used to compute the traffic index (TI) defined by Equation (25) as recommended by Caltrans (Caltrans, 2008). The lane distribution factor is assigned the value of one for two-lane highways.

$$\mathrm{TI} = 9.0 \times \left(\frac{\mathrm{ESAL} \times \mathrm{LDF}}{10^6}\right)^{0.119}. \tag{25}$$

The pavement structure associated with the roadway sample consists of two layers, namely HMA surface and aggregate base. The original structural capacity, $\mathrm{GE}_1(0)$, associated with the asphalt concrete layer is estimated using Equation (26) with the resistance value (R) assumed to be 80 for the aggregate base. Equation (12b) has been used to estimate the asphaltic structural capacity for overlaid pavement, $\mathrm{GE}_1(t)$, required 10 years after reconstruction (t) using the same calibration constants estimated for the arterial sample problem (i.e. $A = 1.4$, $B = 1.2$). The overlay structural capacity, $\mathrm{GE}_0(t)$, is then calculated using Equation (18b) with the corresponding overlay thickness, $h_0(t)$, determined from Equation (19b). Table 3 indicates that the overlay thickness increases as the remaining strength factor decreases, but the estimated overlay thicknesses are higher than expected for low volume roads. This can mainly be attributed to using the calibration constants originally estimated for a major arterial. Table 3 also provides overlay thicknesses computed assuming 1.2 $\mathrm{TP_R}$ as required by Equation (14), which results in about 18–24% higher overlay thicknesses. This latter case can only be justified when considering the sample roads with low distress ratings, an indication of inferior performance.

$$\mathrm{GE} = 0.0032(\mathrm{TI})(100 - R). \tag{26}$$

Table 4 provides sample backward calculations of the calibration constant, $C(j)$, estimated based on the Caltrans design method. Practical overlay thicknesses, $h''_0(t,j)$, have been assigned to the road sample mainly relying on the remaining strength factor, $F(t)$, and applying experience and engineering judgement. Table 4 also provides the original asphalt layer thickness, $D_1(j)$, with the corresponding gravel factor, $\mathrm{Gf}_1(j)$, computed using Equation (27) as recommended by the Caltrans design manual (Caltrans, 2008):

$$\mathrm{Gf}_1 = \frac{5.67}{(\mathrm{TI})^{1/2}}, \quad D_1 \leq 0.5\mathrm{ft},$$

Table 4. Sample backward calculations of the calibration constant, $C(j)$, based on the Caltrans method assuming 10-year rehabilitation scheduling time ($t = 10$ years).

Road j	$D_1(j)$ (cm)	$F(j)$	$\mathrm{Gf}_1(j)$	$h''_0(t,j)$ (cm)	$\mathrm{GE}_1(0,j)$	$\mathrm{GE}''_0(t,j)$	$\mathrm{GE}''_1(t,j)$	$C(j)$	$h_0(t,j)$ (cm)	$E_0(t,j)$[a] (cm)
1	8.0	0.775	1.98	3.0	0.528	0.190	0.599	0.322	3.14	– 0.14
2	7.5	0.715	2.02	3.5	0.505	0.222	0.583	0.366	3.48	0.02
3	8.5	0.656	1.96	4.0	0.555	0.253	0.617	0.270	4.34	– 0.34
4	7.0	0.644	2.09	4.0	0.488	0.253	0.567	0.383	3.91	0.09
5	7.5	0.623	2.01	4.5	0.502	0.285	0.598	0.446	4.19	0.31
6	8.5	0.571	1.94	5.0	0.550	0.317	0.631	0.350	5.04	– 0.04
7	7.0	0.562	2.05	5.0	0.478	0.317	0.586	0.519	4.45	0.55
8	8.5	0.505	1.93	5.5	0.547	0.348	0.624	0.336	5.58	– 0.08
9	8.5	0.487	1.95	5.5	0.552	0.348	0.617	0.284	5.79	– 0.29
10	8.5	0.441	1.94	6.0	0.550	0.380	0.623	0.318	6.17	– 0.17

[a]The overlay error (cm) estimated as the difference between the known practical overlay thickness, $h''_0(t,j)$, and the one predicted, $h_0(t,j)$, using 0.359 average $C(j)$ value.

$$Gf_1 = 7.0 \times \frac{D_1^{1/3}}{(TI)^{1/2}}, \quad D_1 > 0.5\text{ft}. \tag{27}$$

The original asphaltic structural capacity, $GE_1(0,j)$, for the jth road is then computed using Equation (28a), while the overlay structural capacity, $GE''_0(t,j)$, is determined from Equation (28b). The asphaltic structural capacity associated with overlaid pavement 10 years after reconstruction, $GE''_1(t,j)$, and the corresponding calibration constant, $C(j)$, have been estimated using Equation (22) with the results provided in Table 4. The constant $C(j)$ has ranged from 0.270 to 0.519 with 0.359 average value. The average $C(j)$ value is considerably lower than the value of ($A/B = 1.4/1.2 = 1.167$) derived for the major arterial and concluded to be ineffective in estimating the overlay design thickness for the local road sample. The average $C(j)$ value has been used to recalculate the overlay thickness (i.e. predicted value) using Equations (12b), (18b), and (19b) with the results provided in Table 4. The difference between the known practical overlay thickness, $h''_0(t,j)$, and the predicted value, $h_0(t,j)$, is defined as the overlay error, $E_0(t,j)$. According to Table 4, the maximum overlay error is 0.55 cm and the average absolute error is 0.20 cm.

$$GE_1(0,j) = \frac{D_1(j)}{30 \times Gf_1(j)}, \tag{28a}$$

$$GE''_0(t,j) = \frac{1.9 \times h''_0(t,j)}{30}. \tag{28b}$$

Therefore, it can be concluded that the use of the average $C(j)$ value has been successful in predicting the overlay thicknesses for the local road sample. The second calibration constant $D(j)$ can similarly be estimated using Equation (24) if improved pavement deterioration rates are to be deployed in the case of inferior performance.

5. Conclusions and recommendations

The sample overlay thicknesses predicted for the urban arterial seem to be appropriate and in line with the general practice when considering the case of superior performance. The sample results have also indicated a strong correlation between the predicted overlay thickness and rehabilitation scheduling time considering both types of pavement performance. The SN and GE as deployed by the AASHTO and Caltrans design methods have proven to be effective in yielding compatible overlay design thicknesses. The use of the calibration constants associated with the urban arterial has overestimated the overlay design thicknesses for the local road sample. However, the backward approach has effectively been used to estimate the calibration constants $C(j)$ for the road sample mainly relying on original pavement design data and assumed practical overlay thicknesses. The average $C(j)$ value, when applied to the local road sample, has resulted in good estimates of the overlay thicknesses with the corresponding errors being reasonably low. Therefore, it is concluded that an average $C(j)$ value can be developed for each roadway class (i.e. pavement category) provided it is associated with similar traffic loading conditions and materials characteristics.

The successful application of the developed Empirical-Markovian model is greatly influenced by the choice of the appropriate model exponents. As outlined earlier, the minimisation of the SSE can be carried out using a simplified trial and error approach that would lead to reliable estimates of the model exponents at the project level (Abaza, 2017). Alternatively, the proposed backward approach can be used at the network level if original pavement design data and historical rehabilitation records are available. The backward approach is much simpler to use in estimating the model exponents and does not require any stochastic parameters such as the state and transition probabilities. In any case, it is recommended that highway agencies develop a unique set of model exponents for each pavement category with similar traffic loading conditions and materials properties. The proposed overlay model can also be calibrated against other overlay approaches similar to what has been done in the presented road sample wherein the overlay thicknesses are assigned based on experience and engineering judgment (i.e. prescription method). Finally, the developed Empirical-Markovian model mainly deploys pavement strength indicators derived from empirical design methods, namely the AASHTO and Caltrans methods. However, for further research, the author will be investigating the possibility of incorporating strength indicators derived from the Empirical-Mechanistic design approach.

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