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# Optimal Empirical-Markovian approach for assessment of potential pavement rehabilitation strategies at the project level

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An optimal Empirical-Markovian approach is presented for the assessment of potential rehabilitation strategies as applied to flexible pavement at the project level. A simplified empirical model is proposed to predict the heterogeneous transition probabilities associated with rehabilitated pavement based on the corresponding values associated with original pavement, future traffic loading, and modified structural capacity. Therefore, the performance of rehabilitated pavement can directly be controlled by the modified structural capacity. The modified structural capacity associated with a particular rehabilitation strategy is defined using the modified layer coefficients with the corresponding modified structural numbers are used in the empirical model to predict the relevant heterogeneous transition probabilities. These probabilities are then used to estimate the annual distress ratings (DRs) associated with each potential rehabilitation strategy for a specified analysis period. The optimal assessment of potential rehabilitation strategies is carried out using the cost-effectiveness ratio defined as the ratio of the life-cycle average DR to the life-cycle cost. The life-cycle cost can include cost items such as initial construction cost, routine maintenance cost, and major rehabilitation cost. The optimal rehabilitation strategy is the one associated with the highest cost-effectiveness ratio. The presented case study has indicated the effectiveness of the proposed optimal approach in developing and yielding dependable optimal rehabilitation strategies.

**Keywords:** flexible pavement; pavement rehabilitation; pavement performance; pavement design; heterogeneous Markov chains; pavement management

## 1. Introduction

### 1.1. *General background*

Pavement management has traditionally dealt with establishing optimal maintenance and rehabilitation (M&R) schedules at the network level. Pavement management requires periodical collection of pavement distress data, incorporation of a reliable performance prediction model, definition of potential M&R strategies, and development of an effective decision-making criterion to be optimised using an appropriate optimisation technique. Pavement management at the network level mainly focuses on selecting and prioritising pavement projects over a specified analysis period (Gurganus & Gharaibeh, 2012; Jorge & Ferreira, 2012; Medury & Madanat, 2014; Saliminejad & Perrone, 2015; Shahin, 2005; Torres-Machí, Chamorro, Videla, Pellicer, & Yepes, 2013). However, pavement management at the project level mainly focuses on identifying the appropriate rehabilitation strategy to be applied at the optimal scheduling time. It essentially involves the same steps required at the network level but the problem is considered much simpler

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to solve. It is also expected that the M&R strategies recommended at the project level to differ from the ones suggested at the network level. Therefore, it is typically recommended that pavement management at the project level be performed once a pavement project is ready to go for design prior to construction (Cirilovic, Mladenovic, & Queiroz, 2014; Mohajerani & HE, 2014; Priya, Srinivasan, & Veeraragavan, 2008; Santos, Ferreira, & Flintsch, 2015).

Pavement performance prediction is a key element in pavement management modelling which is vitally needed to forecast the pavement future conditions. There are typically two general types of performance prediction models, namely deterministic and probabilistic with the latter being the most widely used one. Several versions of the probabilistic model were used by different researchers to predict pavement performance with the most popular are the Markovian-based ones (Abaza, 2016; Butt, Shahin, Carpenter, & Carnahan, 1994; Durango & Madanat, 2002; Hong & Wang, 2003; Lethanh & Adey, 2013; Mandiartha, Duffield, Thompson, & Wigan, 2012; Meidani & Ghanem, 2015). Different types of Markov chain were used by several researchers including discrete-time Markov chain, discrete-time semi-Markov chain, exponential hidden Markov chain, Poisson hidden Markov chain, random Markov chain, and recurrent Markov chain (Abaza, 2015; Lethanh & Adey, 2013; Lethanh, Kaito, & Kobayashi, 2014; Meidani & Ghanem, 2015; Yang, Lu, Gunaratne, & Dietrich, 2006; Zhang & Gao, 2012). While these researchers applied different forms of the Markov model, they all reported a good degree of success in mainly predicting the performance of original pavements. In particular, the discrete-time Markov chain with heterogeneous transition probabilities has proven to be effective in predicting the performance of original pavement (Abaza, 2015). However, limited work has been done to predict the performance of rehabilitated pavement. A main advantage of the proposed Empirical-Markovian approach is its ability to predict the heterogeneous transition probabilities for rehabilitated pavement from the corresponding ones associated with original pavement. Another advantage is its efficacy in incorporating the expected performance of potential rehabilitation strategies to become an integrated part of the prediction decision-making process.

### 1.2. *Research objectives*

The Empirical-Markovian approach is essentially proposed to predict the heterogeneous transition probabilities associated with rehabilitated pavement as a function of the corresponding ones for original pavement, structural capacity factor, and traffic load factor. The two deployed factors are defined as ratios of the structural capacities and traffic load applications associated with both original and rehabilitated pavements. The structural capacity of a particular rehabilitation strategy is represented by the modified structural number (SN), which can be estimated from the modified layer coefficients (AASHTO, 1993; Huang, 2004). The paper presents simplified models for estimating the modified layer SNs especially in the case of a two-layer pavement structure. The traffic load factor is defined using the 80 kN equivalent single axle load (ESAL) applications.

The proposed optimal assessment approach requires the selection of a limited number of potential rehabilitation strategies to be scheduled at different rehabilitation times. The proposed Empirical-Markovian approach can estimate the long-term performances associated with the selected potential rehabilitation strategies mainly relying on the predicted heterogeneous deterioration transition probabilities. In particular, the life-cycle average distress rating (DR) is estimated for each rehabilitation strategy over a specified analysis period. A cost-effectiveness ratio is used to assess the various selected rehabilitation strategies, which is defined as the ratio of the life-cycle average DR to life-cycle cost. The life-cycle cost can include routine maintenance cost, rehabilitation cost, and initial construction cost. The optimal rehabilitation strategy is the one associated with the highest cost-effectiveness ratio. The main objectives and expected outcomes of this research work can therefore be summarised as follows:

(1) Development of an empirical model that can predict the heterogeneous deterioration transition probabilities associated with rehabilitated pavement from the corresponding values associated with original pavement considering the same pavement structure.
(2) Development of a mechanism to express the structural capacity of the rehabilitated pavement as required by the previously outlined empirical model. It mainly relies on the modified SNs associated with the pavement layers making up the pavement structure. Therefore, the performance of rehabilitated pavement can directly be controlled by varying the structural capacity associated with a particular rehabilitation strategy.
(3) Using a long-term assessment indicator to evaluate the cost-effectiveness of the selected potential rehabilitation strategies. The proposed indicator is simply the earlier outlined cost-effectiveness ratio which needs to be maximised to yield the best rehabilitation strategy.
(4) The selection of the best rehabilitation strategy for a given pavement project will identify the layer thicknesses associated with plain overlay, cold milling and overlay, or reconstruction. Most importantly, it will identify the scheduling time in years associated with the best rehabilitation strategy.

### 1.3. *Overview of discrete-time Markov model*

Several versions of the discrete-time Markovian-based model were used in the literature to model pavement performance (Abaza, 2016; Hong & Wang, 2003; Lethanh & Adey, 2013; Lethanh et al., 2014; Li, Huot, Xie, & Haas, 1995; Li, Xie, & Haas, 1996; Mandiartha et al., 2012; Meidani & Ghanem, 2015). The most popular are the ones deploying either homogeneous or heterogeneous Markov chains (Abaza, 2015). The homogeneous Markov chain assumes steady transition probabilities (i.e. deterioration rates) over time, which does not accurately represent pavement deterioration rates as they do increase over time due to the progressive increase in traffic loading and progressive weakening of the pavement structural capacity. On the other hand, the heterogeneous Markov chain can incorporate a different set of transition probabilities for each transition (i.e. time interval) within an analysis period comprised of ($n$) transitions. Equation (1) provides the heterogeneous Markov model used to predict the state probabilities after ($n$) transitions, $Q_i^{(n)}$, mainly replying on the initial state probabilities, $Q_i^{(0)}$, and ($n$) transition probability matrices, $P(k)$. The state probabilities are represented by row vectors and they denote the proportions of pavement that exist in the various condition states at a specified future time. Naturally, the sum of the state probabilities at a given transition must equal one with the initial values assumed equal to (1, 0, 0, . . . , 0) for new and rehabilitated pavements provided the number of deployed condition states is adequately selected. A maximum of 10 condition states would be adequate to satisfy this assumption (Abaza, 2015).

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

where,

$$Q^{(n)} = (Q_1^{(n)}, Q_2^{(n)}, \ldots, Q_m^{(n)}),$$
$$Q^{(0)} = (Q_1^{(0)}, Q_2^{(0)}, \ldots, Q_m^{(0)}).$$

The heterogeneous transition probability matrix is $(m \times m)$ square matrix with $(m)$ being the number of deployed condition states. Equation (2) presents $(10 \times 10)$ special form of the transition matrix wherein two transitions are only allowed for each condition state, namely either remaining in the same state $(i)$ with $P(k)_{i,i}$ probability or transiting into the next worst state $(i+1)$ with $P(k)_{i,i+1}$ probability. Abaza (2015) reported the use of only two transitions is a valid assumption provided the number of deployed condition states and transition length are appropriately selected. It was reported that a Markov chain with 10 condition states $(m)$ and 1-year transition length are proper values to satisfy this assumption. The matrix entries below the main diagonal represent the pavement improvement rates which vanish in the absence of maintenance and rehabilitation works as indicated by Equation (2). However, the $P(k)_{i,i+1}$ transition probabilities denote the pavement deterioration rates which are vital in predicting pavement performance. The sum of any row in the transition probability matrix must add up to one.

$$P(k) = \begin{pmatrix} P(k)_{1,1} & P(k)_{1,2} & 0 & 0 & 0 & \cdots & 0 \\ 0 & P(k)_{2,2} & P(k)_{2,3} & 0 & 0 & \cdots & 0 \\ 0 & 0 & P(k)_{3,3} & P(k)_{3,4} & 0 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & \cdots & 0 & P(k)_{9,9} & P(k)_{9,10} \\ 0 & 0 & 0 & 0 & \cdots & 0 & P(k)_{10,10} \end{pmatrix}. \tag{2}$$

While Markov chain is typically applied at the network level as part of pavement management, it can still mathematically be used at the project level. The transition probabilities are generally estimated from distress data for individual pavement classes making up the pavement network. A pavement class is typically made up of pavement projects with similar materials characteristics and traffic loading conditions, thus expected to exhibit similar long-term performances. However, it is more reliable if the transition probabilities are estimated at the project level as they would better represent the deterioration mechanism of individual projects.

Pavement performance is typically defined using an appropriate pavement condition indicator as a function of service time or accumulated traffic load applications. Among the most popular pavement condition indicators are the present serviceability index, pavement condition index, and international roughness index. Abaza (2016) used a similar indicator called pavement DR which can be predicted from the state probabilities estimated using the outlined heterogeneous Markov model. Equation (3) can be used to predict the distress rating at the $k$th transition, DR($k$), for an analysis period comprised of $(n)$ transitions at the project level. The DR is predicted from the product sum of the state mean DRs $(C_i)$ and state probabilities, $Q_i^{(k)}$, considering a Markov chain with 10 condition states. A DR scale of 100 points is used with each condition state defined by equal 10-point DR range with the state mean DR is being represented by the middle value of the corresponding range as indicated by Equation (3). It is to be noted that higher DR values denote better pavements with the maximum and minimum predicted DR values are being equal to 95 and 5 which correspond to new and totally damaged pavements, respectively.

$$\mathrm{DR}(k) = \sum_{i=1}^{10} C_i \times Q_i^{(k)} \quad (k = 0, 1, 2, \ldots, n), \tag{3}$$

where,

$$Q^{(k)} = \begin{cases} Q_1^{(k)}, & 90 < \mathrm{DR} \le 100, \quad C_1 = 95 \\ Q_2^{(k)}, & 80 < \mathrm{DR} \le 90, \quad C_2 = 85 \\ Q_3^{(k)}, & 70 < \mathrm{DR} \le 80, \quad C_3 = 75 \\ \vdots & \vdots \qquad\qquad\qquad \vdots \\ Q_{10}^{(k)}, & 0 \le \mathrm{DR} \le 10, \quad C_{10} = 5 \end{cases}.$$

The estimation of one set of state probabilities requires one cycle of pavement distress assessment while two consecutive cycles are needed to obtain an estimate of one set of transition probabilities. A flexible pavement project is typically divided into a number of small pavement sections which are individually surveyed for prevailing defects. Abaza (2016) proposed simple models for estimating the observed DR for each section mainly relying on the two most significant pavement defects, namely cracking and deformation. Equation (4) presents an example of such models wherein the pavement section is surveyed for localised defects.

$$\mathrm{DR} = \left( \frac{3A_S - \sum_i \mathrm{SF}_{C_i} A_{C_i} - \sum_i \mathrm{SF}_{D_i} A_{D_i}}{3A_S} \right) \times 100, \tag{4}$$

where,

$$\sum_i \mathrm{SF}_{C_i} A_{C_i} + \sum_i \mathrm{SF}_{D_i} A_{D_i} \le 3A_S,$$

$$\sum_i A_{C_i} + \sum_i A_{D_i} \le A_s,$$

$A_S$ = entire surface area of the pavement section (m$^2$), $\mathrm{SF}_C$ = severity factor associated with a localised cracked area taken on the values of 1, 2, or 3 for low, medium, or high severity, respectively, $A_C$ = localised cracked area (m$^2$), $\mathrm{SF}_D$ = severity factor associated with a localised deformed area taken on the values of 1, 2, or 3 for low, medium, or high severity, respectively, and, $A_D$ = localised deformed area (m$^2$).

The surveyed pavement sections are then assigned to the various deployed condition states according to the DR ranges defined in Equation (3). Abaza (2016) proposed Equations 5(a) and 5(b) to estimate the 1st transition ($k = 1$) initial and terminal deterioration transition probabilities, $P(1)_{1,2}$ and $P(1)_{m-1,m}$, deploying the numbers of pavement sections assigned to the relevant condition states as obtained from two consecutive cycles of pavement distress assessment. Equation 5(c) is used to the estimate the initial state probabilities, $Q_i^{(0)}$.

$$P(1)_{1,2} = \frac{N_1^{(0)} - N_1^{(1)}}{N_1^{(0)}}, \quad N_1^{(0)} \ge N_1^{(1)}, \tag{5a}$$

$$P(1)_{m-1,m} = \frac{N_m^{(1)} - N_m^{(0)}}{N_{m-1}^{(0)}}, \quad N_m^{(1)} \ge N_m^{(0)}, \tag{5b}$$

$$Q_i^{(0)} = \frac{N_i^{(0)}}{N} \quad (i = 1, 2, \ldots, m), \tag{5c}$$

where, $N_i^{(0)}$ = initial number of pavement sections in state ($i$), $N_i^{(1)}$ = number of pavement sections in state ($i$) after one transition, and $N$ = total number of surveyed pavement sections.

## 2. Methodology

An Empirical-Markovian approach for estimating the heterogeneous transition probabilities for rehabilitated pavement along with an optimal approach for the assessment of potential pavement rehabilitation strategies are presented in this section. The optimal approach takes into consideration both the pavement life-cycle cost and performance.

### 2.1. *Heterogeneous transition probabilities for original pavement*

The application of the heterogeneous Markov model requires the availability of a transition probability matrix for each transition within the analysis period. Abaza (2015) proposed an empirical model to estimate the required transition probability matrices mainly depending on the present transition probability matrix, $P(k = 1)$, and two main factors affecting pavement deterioration over time, namely traffic load and pavement strength factors as presented in Equation (6). The traffic load factor accounts for the progressive increase in traffic loading over time. It is defined as the ratio of the 80 kN ESAL applications, $W(k + 1)$, expected to take place during the $(k + 1)$th transition to the corresponding value associated with the preceding transition. The future transition probabilities are expected to increase over time as the traffic load factor is expected to be greater than one due to traffic load increases. Similarly, the pavement strength factor accounts for the progressive decrease in pavement strength over time. It is defined as the ratio of the pavement strength, $S(k)$, at the $k$th transition to the corresponding value associated with the proceeding transition. The strength factor is expected to be greater than one in the absence of any maintenance and rehabilitation works, thus, resulting in higher future transition probabilities. Structural capacity is typically used to represent pavement strength when considering applications related to pavement rehabilitation and design. The two most popular empirical-based methods for pavement design used the SN and gravel equivalent (GE) as reliable indicators of pavement structural capacity, respectively (AASHTO, 1993; Caltrans, 2008).

$$P(k+1)_{i,i+1} = P(k)_{i,i+1}\left(\frac{W(k+1)}{W(k)}\right)^{A}\left(\frac{S(k)}{S(k+1)}\right)^{B} \quad (k = 1,\ 2,\ 3,\ \ldots,\ n), \tag{6}$$

where, $W(k)$ = 80 kN ESAL applications expected to travel the pavement during the $k$th transition, and $S(k)$ = pavement structural capacity at the $k$th transition.

The empirical model indicated by Equation (6) can be used to estimate the deterioration transition probabilities, $P(k + 1)_{i,i+1}$, as required by the transition probability matrix defined in Equation (2). The empirical model has two exponents (A and B) which can be estimated from the minimisation of sum of squared errors (SSE) wherein the error is defined as the difference between the observed and predicted DRs. The predicted DR is estimated using Equation (3) based on the state probabilities derived from the Markov model presented in Equation (1). Abaza (2015) provided typical values for the exponents (A and B) considering two distinct types of pavement performance as explained later.

### 2.2. *Heterogeneous transition probabilities for rehabilitated pavement*

In this paper, a model similar to the one presented in Equation (6) is proposed to estimate the deterioration transition probabilities associated with rehabilitated pavement as defined in Equation (7). The formulation of the new model is based on the main assumption that both the original and rehabilitated pavements will exhibit similar performance trends as represented by their corresponding performance curves shown in Figure 1. In particular, it is assumed that the performance curve segment associated with rehabilitated pavement between the two transitions $(n + k)$ and

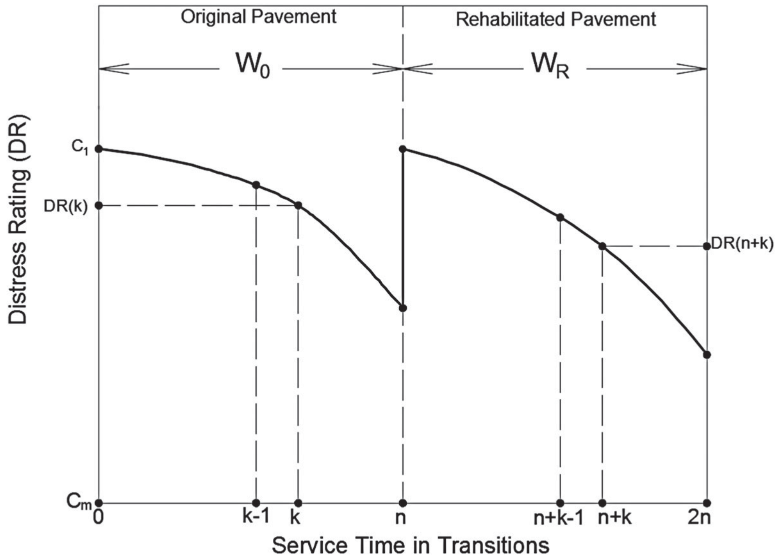


Figure 1. Typical life-cycle pavement performance curve and rehabilitation scheduling plan.

$(n + k - 1)$ to be similar in trend to the corresponding one associated with original pavement between the two transitions $(k)$ and $(k - 1)$. According to Figure 1, it is defined that pavement rehabilitation is scheduled to take place at the nth transition, which is the same as the terminal service time for original pavement.

The traffic load factor is defined as the ratio of the traffic load applications associated with rehabilitated pavement at the $(n + k)$th transition to the corresponding value associated with original pavement at the $(k)$th transition as depicted in Figure 1. Similarly, the pavement strength factor is defined as the ratio of the pavement strength associated with original pavement at the $(k)$th transition to the corresponding value associated with rehabilitated pavement at the $(n + k)$th transition. The strength factor used in Equation (7) can be less than one if the structural capacity associated with rehabilitated pavement is larger than the corresponding one associated with original pavement, which can contribute to improving the performance of rehabilitated pavement or at least counterbalance the impact of increased traffic load applications. Also, environmental and weather conditions can affect pavement deterioration, but they are neglected in Equation (7) because they are expected to be similar for both original and rehabilitated pavements considering the same pavement structure.

$$P(n+k)_{i,i+1} = P(k)_{i,i+1}\left(\frac{W(n+k)}{W(k)}\right)^{A}\left(\frac{S(k)}{S(n+k)}\right)^{B} \quad (k = 1, 2, \ldots, n), \tag{7}$$

Where, $W(n + k) =$ 80 kN ESAL applications expected to travel the rehabilitated pavement during the $(n + k)$th transition, $W(k) =$ 80 kN ESAL applications expected to travel the original pavement during the $k$th transition, $S(n + k) =$ structural capacity associated with rehabilitated pavement at the $(n + k)$th transition, and $S(k) =$ structural capacity associated with original pavement at the kth transition.

Equation (7) can be used to estimate the deterioration transition probabilities, $P(n + k)_{i,i+1}$, associated with rehabilitated pavement based on the corresponding equivalent values, $P(k)_{i,i+1}$, associated with original pavement for a service life of $(n)$ transitions. Therefore, according to Equation (7), the service life for rehabilitated pavement is considered to be the same as the one associated with original pavement (i.e. $n$ transitions). The predicted deterioration transition probabilities for rehabilitated pavement can then be used to estimate the corresponding distress ratings, $\mathrm{DR}(n + k)$, using Equations (1) and (3).

#### 2.2.1. *Estimation of traffic load factor*

A simplified expression for the traffic load factor, $F_L(n)$, used in Equation (7) can be derived as presented in Equation (8). It is essentially derived based on the first-year 80 kN single axle load applications ($W_f$) multiplied by the future economic factor, $F(k)$, considering the two equivalent transitions $(n + k)$ and $(k)$. The future economic factor deployed is simply the one used to convert a present value to a future one. According to Equation (8), the traffic load factor is only dependent on the uniform annual traffic growth rate ($r$), in decimal form, and rehabilitation scheduling time ($n$) in transitions.

$$F_L(n) = \frac{W(n+k)}{W(k)} = \frac{W_f \times F(n+k)}{W_f \times F(k)} = (1+r)^n, \tag{8}$$

where,

$$F(k) = (1+r)^k.$$

The same expression for the traffic load factor can be obtained based on the ratio of the accumulated 80 kN single axle load applications ($W_R$) associated with rehabilitated pavement to the corresponding value ($W_o$) associated with original pavement as indicated by Equation (9). This essentially states that the ratio of traffic load applications associated with the two equivalent transitions $(n + k)$ and $(k)$ is constant and equals to the ratio of $(W_R/W_o)$. The accumulated load applications are computed from multiplying the first-year load applications ($W_f$) by the corresponding traffic growth factor ($G$). The traffic growth factor used is the same one recommended by the Asphalt Institute (Asphalt Institute, 1999).

$$F_L(n) = \frac{W_R}{W_o} = \frac{W_f \times G(2n) - W_f \times G(n)}{W_f \times G(n)} = (1+r)^n, \tag{9}$$

where,

$$G(n) = \frac{(1+r)^n - 1}{r}.$$

#### 2.2.2. *Estimation of pavement strength factor*

The pavement strength factor can be estimated using an appropriate relative strength indicator such as the outlined SN used by AASHTO in its empirical-based pavement design method (AASHTO, 1993). Additionally and as a simplification, it is assumed that the strength change rate for original pavement to be equal to the corresponding one associated with rehabilitated pavement considering the two transitions $(k)$ and $(n + k)$. This results in the strength factor, $F_S(n)$, to be equal to the ratio of design structural number ($\mathrm{SN_o}$) associated with original pavement to the corresponding value, $\mathrm{SN_R}(n)$, associated with rehabilitated pavement as indicated by Equation (10), an outcome that is similar to the one obtained for the load factor.

$$F_s(n) = \frac{S(k)}{S(n+k)} = \frac{\mathrm{SN}(k)}{\mathrm{SN}(n+k)} = \frac{\mathrm{SN_o}}{\mathrm{SN_R}(n)}. \tag{10}$$

The final empirical model for predicting the deterioration transition probabilities for rehabilitated pavement can then be represented by Equation (11) using the SN as an indicator of pavement strength. Generally, it is required to design rehabilitated pavement with SN higher than the one associated with original pavement to compensate for the impact of increased traffic load applications while maintaining similar performance trends.

$$P(n+k)_{i,i+1} = P(k)_{i,i+1}[(1+r)^n]^A \left(\frac{\mathrm{SN_o}}{\mathrm{SN_R}(n)}\right)^B \quad (k = 1,\ 2,\ \ldots,\ n). \tag{11}$$

The SN associated with original pavement is generally known but the one associated with rehabilitated pavement needs to be estimated based on a proposed potential rehabilitation strategy as outlined next.

### 2.2.3. *Estimation of modified structural capacity*

The pavement structural capacity can be defined using the SN as outlined earlier. The structural number associated with original pavement ($\mathrm{SN_o}$) is determined from the product sum of the original layer coefficients ($a_j$) and original layer thicknesses ($D_j$) as presented in Equation (12a) (AASHTO, 1993). Similarly, the structural number for rehabilitated pavement, $\mathrm{SN_R}(n)$, is equal to the sum of modified structural numbers, $\mathrm{SN}_j(n)$, associated with all layers making up the pavement structure as presented in Equation (12b). The modified SN for the $j$ th layer is computed as the product of the modified layer coefficient, $a'_j(n)$, and modified layer thickness, $D'_j(n)$. There are generally two approaches for estimating the modified layer coefficients mainly depending on either destructive or non-destructive testing of pavement (AASHTO, 1993; Huang, 2004).

$$\mathrm{SN_o} = \sum_j \mathrm{SN}_j = \sum_j a_j \times D_j, \tag{12a}$$

$$\mathrm{SN_R}(n) = \sum_j \mathrm{SN}_j(n) = \sum_j a'_j(n) \times D'_j(n). \tag{12b}$$

The most popular flexible pavement structure consists of two layers, namely asphalt concrete surface and aggregate base. The potential rehabilitation strategies typically involve plain overlay, cold milling and overlay, or complete removal/replacement of existing asphalt layer with adjustment of existing aggregate base thickness. Therefore, the modified structural numbers, $\mathrm{SN}_1(n)$ and $\mathrm{SN}_2(n)$, associated with the asphalt and aggregate layers can be determined using Equations (13a) and (13b), respectively. In Equation (13a), the cold milling thickness ($D_\mathrm{m}$) is subtracted from the existing asphalt layer thickness ($D_1$) with the outcome multiplied by the modified layer coefficient, $a'_1(n)$, to account for asphalt strength reduction at the time of rehabilitation ($n$). An addition is then made to account for the SN associated with the overlay as obtained from multiplying the overlay thickness, $\Delta D_1(n)$, by the corresponding overlay coefficient, $a_1(n)$.

$$\mathrm{SN}_1(n) = \frac{1}{2.5}[a'_1(n) \times (D_1 - D_\mathrm{m}) + a_1(n) \times \Delta D_1(n)], \tag{13a}$$

$$SN_2(n) = \frac{1}{2.5}[a'_2(n) \times D_2 + a_2(n) \times \Delta D_2(n)]. \tag{13b}$$

The modified SN for the aggregate base is similarly computed using Equation (13b) with $\Delta D_2(n)$ representing the change in base thickness in the case of complete removal of existing asphalt layer. All layer thicknesses used in Equations (13a) and (13b) are in centimetres; therefore, the entire equations are divided by 2.5 as layer thicknesses have to be in inches according to the AASHTO design method (AASHTO, 1993). While the literature provides several approaches for estimating the modified layer coefficients, the author proposes a simplified solution for estimating the modified layer coefficient for asphalt concrete as presented in Equation (14).

$$a_1'(n) = a_1 \times F_1(n) = a_1 \frac{\mathrm{DR}(n)}{C_1}. \tag{14}$$

The original asphalt layer coefficient ($a_1$) is multiplied by the strength reduction factor, $F_1(n)$, to yield an estimate of the modified asphalt layer coefficient, $a_1'(n)$. The asphalt strength reduction factor is estimated as the ratio of the distress rating associated with the nth transition, DR($n$), to the maximum distress rating, namely ($C_1$) as defined in Equation (3). The aggregate base layer generally experiences minor strength reduction and can be neglected; however, destructive/non-destructive testing can be used to estimate the corresponding modified layer coefficient. Once the modified SNs are estimated for all layers making up the pavement structure, then Equation (11) can be used to predict the deterioration transition probabilities associated with rehabilitated pavement. However, if it is assumed that the asphalt layer is the main contributor of pavement deterioration and the impact of underlying layers is neglected, then Equation (15) can be used in lieu of Equation (11).

$$P(n+k)_{i,i+1} = P(k)_{i,i+1}[(1+r)^n]^A \left(\frac{\mathrm{SN}_1}{\mathrm{SN}_1(n)}\right)^B \quad (k = 1,\ 2,\ \ldots,\ n). \tag{15}$$

Alternatively, Equation (16) can be used wherein the impact of the underlying layers is somewhat considered in the estimation of the transition probabilities compared to Equation (15). This is because the value of the strength factor used in Equation (16) is mathematically higher than the corresponding value deployed in Equation (15). Therefore, Equation (16) will relatively yield higher transition probabilities compared to Equation (15), an indication that the underlying layers have partially contributed to pavement deterioration. Equation (16) thus provides a more general and realistic model if all relevant data are available.

$$P(n+k)_{i,i+1} = P(k)_{i,i+1}[(1+r)^n]^A \left(\frac{\mathrm{SN_o}}{\mathrm{SN_o} - \mathrm{SN}_1 + \mathrm{SN}_1(n)}\right)^B \quad (k = 1,\ 2,\ \ldots,\ n). \tag{16}$$

The performance curve portion associated with original pavement as shown in Figure 1 can be developed using the distress ratings, DR($k$), estimated from the deterioration transition probabilities, $P(k)_{i,i+1}$, predicted using Equation (6) as outlined earlier. Similarly, the performance curve portion associated with rehabilitated pavement can be generated using the distress ratings, DR($n + k$), estimated based on the deterioration transition probabilities, $P(n + k)_{i,i+1}$, predicted using either Equations (11), (15), or (16).

### 2.2.4. *Estimation of empirical model exponents*

The two exponents (A and B) associated with the empirical models presented for estimating the deterioration transition probabilities for both original and rehabilitated pavements can be estimated using different techniques provided relevant pavement distress records are available. Abaza (2015) applied the minimisation of SSEs as outlined earlier to estimate the two exponents for original pavement. This study indicated that the value ranges of (A and B) are generally (1.0–2.0) for performance with increasingly higher deterioration transition probabilities (i.e. $P_{1,2} < P_{2,3} < \cdots < P_{9,10}$), and (0.0–1.0) for performance with decreasingly lower deterioration transition probabilities (i.e. $P_{1,2} > P_{2,3} > \cdots > P_{9,10}$). The exponent values for rehabilitated pavement are expected to be similar provided the corresponding performance trend is also similar. A simpler technique to obtain estimates of the exponent values for rehabilitated pavement is to apply Equation (11) for two rehabilitated projects ($x$ and $y$) with similar materials and traffic characteristics but different input data as required by Equation (11). The outcome is Equations (17a) and (17b) to be simultaneously solved for the two exponents after performing the required linear transformations using the natural logarithmic function.

$$P(n_x + 1)_{i,i+1} = P_x(1)_{i,i+1}[(1 + r)^{n_x}]^{A_i}\left(\frac{\mathrm{SN}_x}{\mathrm{SN}_R(n_x)}\right)^{B_i} \quad (k = 1), \tag{17a}$$

$$P(n_y + 1)_{i,i+1} = P_y(1)_{i,i+1}[(1 + r)^{n_y}]^{A_i}\left(\frac{\mathrm{SN}_y}{\mathrm{SN}_R(n_y)}\right)^{B_i} \quad (k = 1). \tag{17b}$$

According to Equation (17), one estimate of the deterioration transition probabilities associated with each one of the two rehabilitated projects ($x$ and $y$) is required to obtain a distinct set of the exponents ($A_i$ and $B_i$) for the $i$th transition probability. Therefore, if pavement distress data are collected and used to estimate the transition probabilities associated with the two rehabilitated projects at the end of the first transition ($k = 1$), then it would be possible to obtain an estimate of the corresponding exponents. Generally, a pavement network can be divided into a small number of classes and one set of exponents can be developed for each pavement class.

### 2.3. *Optimal assessment of potential rehabilitation strategies*

The reliable assessment of potential rehabilitation strategies requires investigating both the relevant cost and performance. The performance associated with a particular rehabilitation strategy can be assessed using the performance curve derived as outlined earlier. A reliable indicator of pavement performance is the average DR estimated over a specified analysis period. The average distress rating ($\mathrm{DR_{LC}}$) computed over a life-cycle analysis period comprised of ($2n$) transitions is presented in Equation (18), which accounts for the performances of both original and rehabilitated pavements as depicted in Figure 1. The optimal rehabilitation strategy can be considered to be the one associated with the highest life-cycle average DR if performance is to be solely used in the assessment process.

$$\mathrm{DR_{LC}} = \frac{\sum_{k=0}^{n} \mathrm{DR}(k) + \sum_{k=0}^{n} \mathrm{DR}(n + k)}{2(n + 1)}. \tag{18}$$

However, any reliable assessment of potential rehabilitation strategies must also take into consideration the life-cycle cost. There are several cost items that can be considered in a life-cycle analysis including initial construction cost, routine maintenance cost, and major rehabilitation cost. However, the main cost item to be considered is the rehabilitation cost which can also greatly affect routine maintenance cost. Generally, the appropriate rehabilitation strategy mainly depends on the rehabilitation scheduling time; for example, extensive rehabilitation works are required at advanced rehabilitation times. Life-cycle cost assessment of potential rehabilitation strategies requires related cost items to be expressed as annual amounts since the rehabilitation scheduling time ($n$) is a variable one, thus, resulting in a variable life-cycle analysis period comprised of ($2n$) transitions. Equation (19) can be used to convert the present value of the initial construction cost ($P_{\mathrm{IC}}$) and rehabilitation cost ($P_{\mathrm{RC}}$) into equivalent annual costs. Rehabilitation cost is essentially a future value but it is typically estimated based on current local prices. The present cost is multiplied by the relevant economic conversion factor, $f(A/P, i, 2n)$, using the uniform annual discount rate ($i$), in decimal form, and an analysis period of ($2n$) transitions. The length of one transition is considered to be equal to one-year time interval, which is also the same as the time length between successive cycles of pavement distress assessment.

$$A_{\mathrm{IC}} \text{ or } A_{\mathrm{RC}} = P_{\mathrm{IC}} \text{ or } P_{\mathrm{RC}} \times f\,(A/P,\ i,\ 2n), \tag{19}$$

where,

$$f\,(A/P,\ i,\ 2n) = \frac{i \times (1+i)^{2n}}{(1+i)^{2n} - 1}.$$

The optimal rehabilitation strategy can be considered to be the one associated with the lowest life-cycle cost, but it may not necessarily be the same one with the highest life-cycle performance. Therefore, the cost-effectiveness ratio ($R_{\mathrm{CE}}$) as defined in Equation (20) can be used as a reliable indicator to account for both life-cycle performance ($\mathrm{DR}_{\mathrm{LC}}$) and life-cycle cost ($A_{\mathrm{LC}}$). The optimal rehabilitation strategy becomes the one associated with the highest cost-effectiveness ratio. Therefore, it is required to maximise the cost-effectiveness ratio considering a number of potential rehabilitation strategies with a variable scheduling time ($n$). The optimal rehabilitation strategy is thus the one applied at the optimal rehabilitation scheduling time.

$$\text{Maximize:} \quad R_{\mathrm{CE}} = \frac{\mathrm{DR}_{\mathrm{LC}}}{A_{\mathrm{LC}}} = \frac{\mathrm{DR}_{\mathrm{LC}}}{A_{\mathrm{IC}} + A_{RC}}. \tag{20}$$

The optimal assessment process begins with selecting a number of potential rehabilitation strategies for different scheduling times. The potential rehabilitation strategies typically include plain overlay, cold milling and overlay, and reconstruction. Reconstruction includes complete removal of the existing asphalt layer, possible adjustment of aggregate base thickness, and placement of new asphalt layer. The optimal rehabilitation scheduling time generally ranges from 5 to 10 years with integer values are to be only selected in the optimal assessment process. The structural number associated with rehabilitated pavement ($\mathrm{SN_R}$) is estimated as outlined earlier with the corresponding deterioration transition probabilities are predicted using either one of the presented empirical models. The deterioration transition probabilities are used to estimate the distress ratings, DR($n + k$), associated with rehabilitated pavement which are then used along with the distress ratings, DR($k$), for original pavement to compute the life-cycle average DR. The life-cycle cost is also estimated using potential cost items and used along with the life-cycle average DR to estimate the cost-effectiveness ratio. A sample presentation is next presented to demonstrate the entire optimal assessment process.

## 3. Sample presentation

A case study is presented in this section that mainly focuses on predicting the performance of a rehabilitated pavement structure using relevant heterogeneous deterioration transition probabilities estimated from the proposed Empirical-Markovian-based model. A number of potential rehabilitation strategies are then investigated for the purpose of identifying the optimal rehabilitation strategy and its optimal scheduling time.

### 3.1. *Performance of original pavement structure*

The sample pavement structure under consideration belongs to a major urban arterial located in the city of Nablus, Palestine, which was designed to withstand 5-million ESAL applications. The pavement structure was constructed using 12-cm high-stability asphalt concrete ($D_1$) and 50-cm aggregate base ($D_2$). The corresponding asphalt structural number ($\mathrm{SN_1}$) is 2.1 obtained using

0.44 original layer coefficient ($a_1$). Abaza (2015) predicted the original pavement performance for this urban arterial using mainly the initial and terminal deterioration transition probabilities ($P_{1,2}$ and $P_{9,10}$) with the remaining transition probabilities estimated from linear interpolation as indicated by Equations (21a) and (21b) considering a Markov chain with 10 condition states. Equation (21a) was used for pavement performance associated with increasingly higher deterioration transition probabilities (superior performance) while Equation (21b) used for performance with decreasingly lower deterioration transition probabilities (inferior performance). The initial and terminal deterioration transition probabilities for the first transition ($k = 1$) were estimated from pavement distress assessment and found to be (0.182 and 0.384) and (0.650 and 0.180) for superior and inferior performance trends, respectively.

$$P(k)_{i,i+1} = P(k)_{1,2} + (i-1)\left(\frac{P(k)_{9,10} - P(k)_{1,2}}{8}\right) \quad (i = 2, 3, \ldots, 8), \tag{21a}$$

where, $P(k)_{1,2} < P(k)_{2,3} < \cdots < P(k)_{9,10}$,

$$P(k)_{i,i+1} = P(k)_{1,2} - (i-1)\left(\frac{P(k)_{1,2} - P(k)_{9,10}}{8}\right) \quad (i = 2,\ 3,\ \ldots, 8), \tag{21b}$$

where: $P(k)_{1,2} > P(k)_{2,3} > \cdots > P(k)_{9,10}$.

Abaza (2015) estimated the model exponents ($A$ and $B$) associated with Equation (6) from the minimisation of SSE and found to be (1.4 and 1.2) and (0.7 and 0.4) for superior and inferior performances, respectively. Equation (6) was then used to predict the heterogeneous deterioration transition probabilities for an analysis period comprised of 20 transitions. One year is considered to be equivalent to one transition. The annual DRs were estimated using Equation (3) with Figure 2 showing the corresponding performance curves generated for the two types of pavement performance. The superior performance was identified over the arterial segments that were built on subgrade with good bearing strength while the inferior one was spotted over segments that were built on subgrade with poor bearing strength.

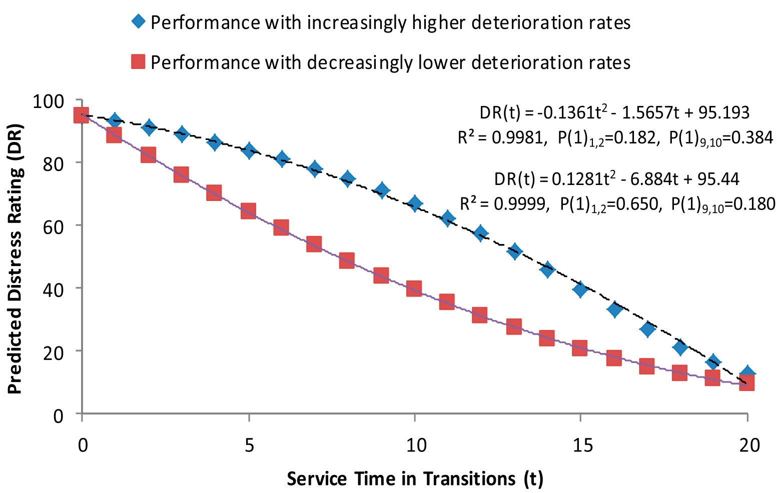


Figure 2. Sample pavement performance curves for original pavement predicted using heterogeneous Markov chain.

Table 1. Sample modified SNs for potential rehabilitation strategies.

| $n$ | DR ($n$) | $F_1$ ($n$) | $D_m$ (cm) | $\Delta D_1(n)$ (cm) | $SN'_1(n)$ |
|---|---|---|---|---|---|
| (a) Superior performance | | | | | |
| 5 | 83.96 | 0.884 | 0 | 2.5 | 2.31 |
| 6 | 80.90 | 0.852 | 0 | 3.0 | 2.33 |
| 7 | 77.56 | 0.816 | 0 | 3.5 | 2.34 |
| 8 | 73.96 | 0.778 | 0 | 4.5 | 2.44 |
| 9 | 70.08 | 0.738 | 0 | 5.5 | 2.53 |
| 10 | 65.98 | 0.695 | 0 | 6.5 | 2.61 |
| (b) Inferior performance | | | | | |
| 5 | 64.22 | 0.676 | 2 | 6.5 | 2.33 |
| 6 | 58.75 | 0.618 | 2 | 7.5 | 2.41 |
| 7 | 53.53 | 0.563 | 3 | 9.0 | 2.47 |
| 8 | 48.57 | 0.511 | 3 | 9.5 | 2.48 |
| 9 | 43.86 | 0.462 | 4 | 10.5 | 2.50 |
| 10 | 39.41 | 0.415 | 4 | 11.5 | 2.61 |

Table 2. Sample cold milling thickness as a function of asphalt strength reduction factor.

| $F_1$ ($n$) | $D_m$ (cm)[a] |
|---|---|
| 1.0–0.7 | 0 |
| 0.7–0.6 | 2 |
| 0.6–0.5 | 3 |
| 0.5–0.4 | 4 |
| 0.4–0.3 | 5 |
| 0.3–0.2 | 6 |
| 0.2–0.0 | –[b] |

[a]Cold milling thickness recommended not to exceed half the existing asphalt thickness.
[b]Complete removal of existing asphalt concrete layer.

### 3.2. *Performance of rehabilitated pavement structure*

The performance of a sample of potential rehabilitation strategies to be applied to the outlined original pavement structure has been considered in this section. The sample potential rehabilitation strategies have been selected to mainly consist of cold milling and overlay. Table 1 provides the selected cold milling thickness ($D_m$) and overlay thickness, $\Delta D_1(n)$, for both types of pavement performance. The cold milling thickness is selected using the tentative guidelines provided in Table 2 mainly depending on the asphalt strength reduction factor, $F_1(n)$, which is computed as defined in Equation (14). The rehabilitation scheduling time ($n$) has been varied from 5 to 10 years with the corresponding distress ratings, DR($n$), estimated using the models provided in Figure 2. The modified asphalt structural number, $SN'_1(n)$, has been calculated using Equation (13a) with the corresponding modified layer coefficient, $a'_1(n)$, computed from Equation (14). The overlay coefficient, $a_1(n)$, is assumed equal to 0.44. The overlay thickness has been selected so there is a consistent gradual increase in the modified asphalt SN to counterbalance the progressive increase in traffic loading with its value at 5-year rehabilitation scheduling time is greater than the 2.1 value associated with the original asphalt layer.

Table 3. Sample initial deterioration transition probabilities for potential rehabilitation strategies with superior performance.

| | | $P_{1,2}(n+k)$ | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | $P_{1,2}(k)$ | $n=5$ | $n=6$ | $n=7$ | $n=8$ | $n=9$ | $n=10$ |
| 1 | 0.182 | 0.200 | 0.206 | 0.214 | 0.212 | 0.211 | 0.212 |
| 2 | 0.197 | 0.216 | 0.223 | 0.231 | 0.229 | 0.229 | 0.230 |
| 3 | 0.208 | 0.228 | 0.236 | 0.244 | 0.242 | 0.241 | 0.242 |
| 4 | 0.220 | 0.241 | 0.249 | 0.258 | 0.256 | 0.255 | 0.256 |
| 5 | 0.233 | 0.256 | 0.264 | 0.273 | 0.271 | 0.271 | 0.272 |
| 6 | 0.246 | | 0.278 | 0.289 | 0.287 | 0.286 | 0.287 |
| 7 | 0.260 | | | 0.305 | 0.303 | 0.302 | 0.303 |
| 8 | 0.275 | | | | 0.320 | 0.319 | 0.320 |
| 9 | 0.290 | | | | | 0.336 | 0.338 |
| 10 | 0.307 | | | | | | 0.358 |

Table 4. Sample terminal deterioration transition probabilities for potential rehabilitation strategies with superior performance.

| | | $P_{9,10}(n+k)$ | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | $P_{9,10}(k)$ | $n=5$ | $n=6$ | $n=7$ | $n=8$ | $n=9$ | $n=10$ |
| 1 | 0.384 | 0.421 | 0.435 | 0.451 | 0.447 | 0.446 | 0.447 |
| 2 | 0.415 | 0.456 | 0.470 | 0.487 | 0.483 | 0.482 | 0.484 |
| 3 | 0.439 | 0.482 | 0.497 | 0.515 | 0.511 | 0.509 | 0.512 |
| 4 | 0.464 | 0.510 | 0.525 | 0.544 | 0.540 | 0.538 | 0.541 |
| 5 | 0.491 | 0.539 | 0.556 | 0.576 | 0.571 | 0.570 | 0.572 |
| 6 | 0.519 | | 0.587 | 0.609 | 0.604 | 0.602 | 0.605 |
| 7 | 0.548 | | | 0.643 | 0.637 | 0.636 | 0.639 |
| 8 | 0.580 | | | | 0.675 | 0.673 | 0.676 |
| 9 | 0.613 | | | | | 0.711 | 0.714 |
| 10 | 0.648 | | | | | | 0.755 |

Equation (15) is then used to predict the heterogeneous deterioration transition probabilities associated with potential rehabilitation strategies, $P(n+k)_{i,i+1}$, as a function of the heterogeneous transition probabilities for original pavement, $P(k)_{i,i+1}$, 3% annual traffic growth rate ($r$), 2.1 original asphalt structural number ($SN_1$), and the corresponding modified asphalt structural numbers, $SN'_1(n)$. Tables 3 and 4 provide the initial and terminal deterioration transition probabilities, respectively, for superior performance while Tables 5 and 6 provide similar results for inferior performance considering a variable rehabilitation scheduling time ($n$). The remaining deterioration transition probabilities are then estimated using Equation (21) for each transition within the analysis period. The results provided in Tables 3 and 4 indicate that the transition probabilities are generally 9–16% higher than the values associated with original pavement whereas the results in Tables 5 and 6 are 6–13% higher, an indication that the deployed rehabilitation strategies are inadequate to maintain the same original deterioration transition probabilities.

The annual DRs associated with potential rehabilitation strategies, DR($n+k$), are then estimated using Equation (3) with results provided in Tables 7 and 8 for superior and inferior performances, respectively. The exponents ($A$ and $B$) of Equation (15) are assumed to take on the same values as the ones estimated for the original pavement structure. This assumption is a reasonable one provided the performance trends for both original and rehabilitated pavements

Table 5. Sample initial deterioration transition probabilities for potential rehabilitation strategies with inferior performance.

| | | $P_{1,2}$ $(n+k)$ | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | $P_{1,2}$ $(k)$ | $n=5$ | $n=6$ | $n=7$ | $n=8$ | $n=9$ | $n=10$ |
| 1 | 0.650 | 0.691 | 0.696 | 0.704 | 0.718 | 0.730 | 0.733 |
| 2 | 0.674 | 0.717 | 0.722 | 0.730 | 0.744 | 0.757 | 0.760 |
| 3 | 0.692 | 0.736 | 0.741 | 0.750 | 0.764 | 0.777 | 0.780 |
| 4 | 0.712 | 0.757 | 0.763 | 0.771 | 0.786 | 0.800 | 0.803 |
| 5 | 0.732 | 0.779 | 0.784 | 0.793 | 0.808 | 0.822 | 0.825 |
| 6 | 0.752 | | 0.806 | 0.815 | 0.830 | 0.845 | 0.848 |
| 7 | 0.773 | | | 0.837 | 0.853 | 0.868 | 0.872 |
| 8 | 0.795 | | | | 0.878 | 0.893 | 0.896 |
| 9 | 0.817 | | | | | 0.918 | 0.921 |
| 10 | 0.840 | | | | | | 0.947 |

Table 6. Sample terminal deterioration transition probabilities for potential rehabilitation strategies with inferior performance.

| | | $P_{9,10}$ $(n+k)$ | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | $P_{9,10}(k)$ | $n=5$ | $n=6$ | $n=7$ | $n=8$ | $n=9$ | $n=10$ |
| 1 | 0.180 | 0.191 | 0.193 | 0.195 | 0.199 | 0.202 | 0.203 |
| 2 | 0.186 | 0.198 | 0.199 | 0.201 | 0.205 | 0.209 | 0.210 |
| 3 | 0.192 | 0.204 | 0.206 | 0.208 | 0.212 | 0.216 | 0.216 |
| 4 | 0 .197 | 0.210 | 0.211 | 0.213 | 0.218 | 0.221 | 0.222 |
| 5 | 0.203 | 0.216 | 0.218 | 0.220 | 0.224 | 0.228 | 0.229 |
| 6 | 0.208 | | 0.223 | 0.225 | 0.230 | 0.234 | 0.234 |
| 7 | 0.214 | | | 0.232 | 0.236 | 0.240 | 0.241 |
| 8 | 0.220 | | | | 0.243 | 0.247 | 0.248 |
| 9 | 0.226 | | | | | 0.254 | 0.255 |
| 10 | 0.233 | | | | | | 0.263 |

Table 7. Sample predicted DRs for potential rehabilitation strategies with superior performance.

| | | DR $(n+k)$ | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | DR($k$) | $n=5$ | $n=6$ | $n=7$ | $n=8$ | $n=9$ | $n=10$ |
| 0 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 |
| 1 | 93.18 | 92.99 | 92.93 | 92.86 | 92.87 | 92.88 | 92.88 |
| 2 | 91.16 | 90.76 | 90.63 | 90.48 | 90.50 | 90.51 | 90.51 |
| 3 | 88.97 | 88.33 | 88.11 | 87.89 | 87.92 | 87.94 | 87.94 |
| 4 | 86.58 | 85.69 | 85.37 | 85.05 | 85.10 | 85.13 | 85.13 |
| 5 | 83.98 | 82.79[a] | 82.37 | 81.95 | 82.00 | 82.04 | 82.04 |
| 6 | 81.15 | | 79.08[a] | 78.54 | 78.61 | 78.65 | 78.65 |
| 7 | 78.05 | | | 74.79[a] | 74.88 | 74.94 | 74.94 |
| 8 | 74.66 | | | | 70.77[a] | 70.84 | 70.85 |
| 9 | 70.93 | | | | | 66.32[a] | 66.33 |
| 10 | 66.84 | | | | | | 61.33[a] |

[a]Terminal distress rating (DR($2n$)).

Table 8. Sample predicted DRs for potential rehabilitation strategies with inferior performance.

| | | DR ($n + k$) | | | | | |
|---|---|---|---|---|---|---|---|
| $k$ | DR ($k$) | $n = 5$ | $n = 6$ | $n = 7$ | $n = 8$ | $n = 9$ | $n = 10$ |
| 0 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 | 95.00 |
| 1 | 88.50 | 88.09 | 88.02 | 87.96 | 87.82 | 87.70 | 87.66 |
| 2 | 82.16 | 81.37 | 81.25 | 81.12 | 80.86 | 80.63 | 80.56 |
| 3 | 76.04 | 74.91 | 74.74 | 74.56 | 74.20 | 73.87 | 73.76 |
| 4 | 70.14 | 68.72 | 68.50 | 68.28 | 67.82 | 67.40 | 67.27 |
| 5 | 64.47 | 62.78[a] | 62.52 | 62.26 | 61.72 | 61.23 | 61.07 |
| 6 | 59.02 | | 56.82[a] | 56.53 | 55.92 | 55.36 | 55.18 |
| 7 | 53.80 | | | 51.07[a] | 50.40 | 49.79 | 49.60 |
| 8 | 48.81 | | | | 45.16[a] | 44.51 | 44.30 |
| 9 | 44.05 | | | | | 39.52[a] | 39.31 |
| 10 | 39.52 | | | | | | 34.60[a] |

[a]Terminal distress rating (DR($2n$)).

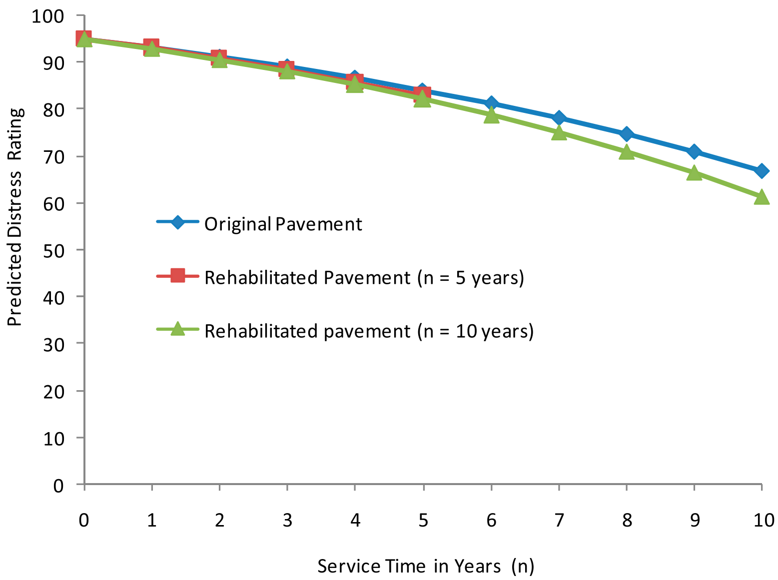


Figure 3. Predicted performance curves for both original and rehabilitated pavements in the case of superior performance ($A = 1.4$, $B = 1.2$).

are similar for the same project. The DRs provided in Tables 7 and 8 have been used to construct Figures 3 and 4 for the cases of 5 and 10 years rehabilitation scheduling times ($n$). It can be concluded from Figures 3 and 4 that the performance trends associated with rehabilitated pavement are similar to the ones associated with original pavement considering both superior and inferior performances, respectively. This conclusion considerably justifies the assumption made for using the same exponent values as of the original pavement. In essence, this provides a means for validating the efficacy of the empirical model presented in Equation (15) especially in the lack of any distress data for rehabilitated pavement. The calibration procedure outlined in Abaza (2015) can be used to obtain revised values of the model exponents ($A$ and $B$) once adequate distress records become available or simply applying Equation (17).

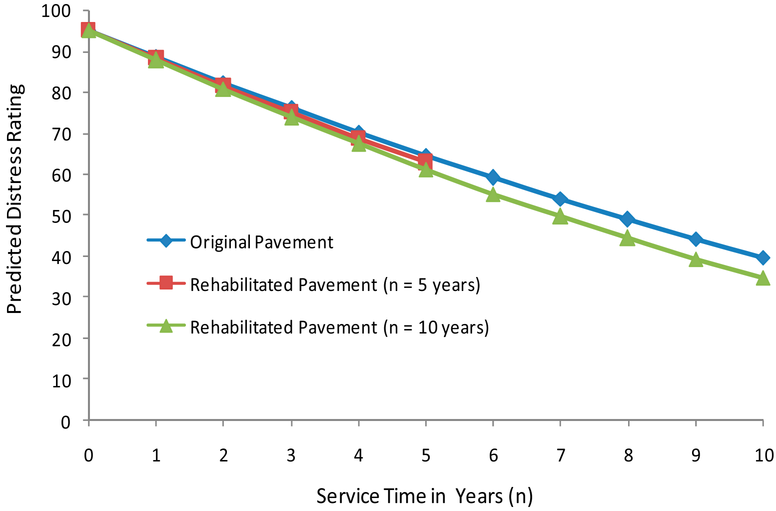


Figure 4. Predicted performance curves for both original and rehabilitated pavements in the case of inferior performance ($A = 0.7, B = 0.4$).

Table 9. Sample cost-effectiveness ratios for potential rehabilitation strategies.

| $n$ | $D_m$ (cm) | $\Delta D_1(n)$ (cm) | $P_{RC}$ | $A_{RC}$ | $A_{IC}$ | $A_{LC}$ | $DR_{LC}$ | $R_{CE}$ |
|---|---|---|---|---|---|---|---|---|
| (a) Superior performance | | | | | | | | |
| 5 | 0 | 2.5 | 5.62[a] | 0.63[a] | 3.06[a] | 3.69[a] | 89.54 | 24.26[b] |
| 6 | 0 | 3.0 | 6.75 | 0.64 | 3.06 | 3.70 | 88.11 | 23.81 |
| 7 | 0 | 3.5 | 7.88 | 0.65 | 3.06 | 3.71 | 86.54 | 23.33 |
| 8 | 0 | 4.5 | 10.12 | 0.75 | 3.06 | 3.81 | 85.02 | 22.31 |
| 9 | 0 | 5.5 | 12.38 | 0.83 | 3.06 | 3.89 | 83.40 | 21.44 |
| 10 | 0 | 6.5 | 14.62 | 0.89 | 3.06 | 3.95 | 81.64 | 20.67 |
| (b) Inferiorperformance | | | | | | | | |
| 5 | 2 | 6.5 | 19.12[a] | 2.13[a] | 3.06[a] | 5.19[a] | 78.93 | 15.21[b] |
| 6 | 2 | 7.5 | 21.38 | 2.02 | 3.06 | 5.08 | 75.87 | 14.94 |
| 7 | 3 | 9.0 | 27.00 | 2.23 | 3.06 | 5.29 | 72.87 | 13.77 |
| 8 | 3 | 9.5 | 28.12 | 2.07 | 3.06 | 5.13 | 69.82 | 13.61 |
| 9 | 4 | 10.5 | 32.62 | 2.18 | 3.06 | 5.24 | 66.85 | 12.76 |
| 10 | 4 | 11.5 | 34.88 | 2.13 | 3.06 | 5.19 | 64.08 | 12.35 |

[a] All cost rates are in the unit of $\$/m^2$.
[b] Optimal rehabilitation strategy based on the highest $R_{CE}$.

### 3.3. *Sample optimal assessment of potential rehabilitation strategies*

Optimal assessment of potential rehabilitation strategies is to be performed using the cost-effectiveness ratio as defined in Equation (20). The present rehabilitation cost rate ($P_{RC}$) mainly includes the cost of cold milling and overlay which is locally estimated at $\$2.25/m^2$ per centimetre of the total cold milling and overlay thickness, $D_m + \Delta D_1(n)$, with results provided in Table 9. It is then converted to an equivalent annual cost ($A_{RC}$) using Equation (19) with 2% annual discount rate ($i$). The present value of the initial construction cost rate ($P_{IC}$) is also locally estimated at $\$50/m^2$. Similarly, it is converted to an equivalent annual cost ($A_{IC}$) of $\$3.06/m^2$ using 20-year service life ($2n$) and equally assigned to all investigated rehabilitation strategies. The annual life-cycle cost rate ($A_{LC}$) is the sum of both the annual rehabilitation cost ($A_{RC}$) and

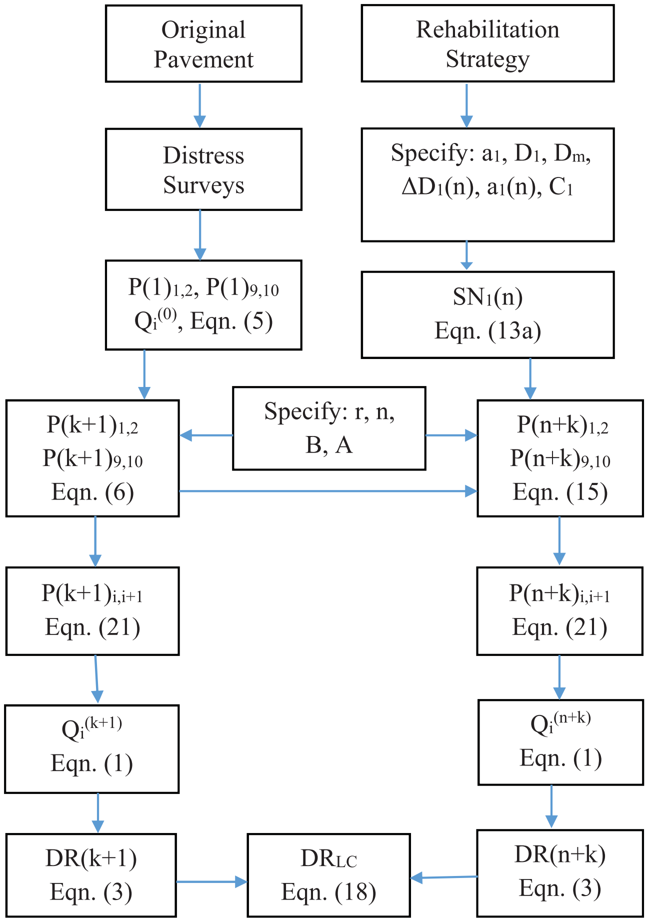


Figure 5. Flowchart depicting the main steps involved in predicting the life-cycle average DR for a particular rehabilitation strategy.

annual initial construction cost ($A_{IC}$) as provided in Table 9. The table also provides the life-cycle average distress rating ($DR_{LC}$) computed for each rehabilitation strategy using Equation (18). The flowchart shown in Figure 5 summarises the main steps involved in the calculation of the life-cycle average distress rating ($DR_{LC}$) for a particular rehabilitation strategy.

The cost-effectiveness ratio ($R_{CE}$) is determined as the ratio of the life-cycle average DR to the life-cycle cost rate. The ($R_{CE}$) values for superior performance are about 65% higher than the corresponding ones for inferior performance. The optimal rehabilitation strategy is the one associated with 5-year scheduling time considering both types of pavement performance, which is essentially the one with the highest life-cycle average DR, but not necessarily the one with the lowest life-cycle cost as in the case of inferior performance. It can also be noted that there is a small variation range in the life-cycle cost rate compared to a larger one in the life-cycle average DR especially in the case of inferior performance. In the case of superior performance, the life-cycle cost rates associated with 5–7 years scheduling times ($n$) are very similar. The cost of routine maintenance, if considered, would be the minimum in the case of 5-year scheduling time and it is expected to exponentially increase with the increase in the rehabilitation scheduling time. The terminal distress rating, DR($2n$), as provided in Tables 7 and 8 can be used in conjunction with the cost-effectiveness ratio to select an optimal rehabilitation strategy. For example, in the case of superior performance, the rehabilitation strategy with 8-year scheduling time can be selected if a terminal DR of 70 is desired.

## 4. Conclusions and recommendations

The presented sample results have indicated that the proposed empirical model can yield reliable estimates of the heterogeneous transition probabilities and consequently the future performance of rehabilitated pavement mainly relying on the transition probabilities associated with original pavement. However, it is important to point out that the presented Empirical-Markovian approach has been verified considering only the typical performance trends shown in Figure 2. The requirements for implementing the proposed Empirical-Markovian approach are also minimal and mainly rely on pavement distress records and original pavement design parameters. Most highway agencies periodically collect pavement distress data which can be used to estimate the required transition probabilities. According to the presented empirical model, if rehabilitation is to be scheduled after ($n$) transitions from the original construction date, then ($n$) estimates of the transition probabilities are required for the original pavement. They can either be estimated from historical pavement distress records or predicted using Equation (6) mainly relying on the first-year transition probabilities (Abaza, 2015, 2016). However, if historical distress records are unavailable for a specific highway, then distress data from a similar new highway can be collected to estimate the first-year transition probabilities, which are then applied to predict the remaining heterogeneous transition probabilities using Equation (6).

The presented sample results have indicated the reliability of the proposed cost-effectiveness ratio in yielding optimal rehabilitation strategies deploying both the life-cycle performance and life-cycle cost. The cost-effectiveness analysis results have indicated that the optimal rehabilitation strategy is the one associated with the highest life-cycle performance, but not necessarily with the lowest life-cycle cost as in the case of inferior performance. They have also indicated that significant improvements in the life-cycle performance can be achieved with mild increases in the life-cycle cost especially in the case of inferior performance. The life-cycle cost can be restricted to the initial construction cost and rehabilitation cost as performed in the sample presentation, but other cost items can be considered if so desired. The life-cycle performance can be represented by the average DR as proposed in this paper or alternatively by the area falling under the life-cycle performance curve. Another main advantage of the proposed Empirical-Markovian approach is that it can effectively account for the structural capacity of the individual rehabilitation strategies as represented by the corresponding modified SNs. In essence, the proposed Empirical-Markovian approach provides a simplified performance–based procedure for the design of flexible pavement. For example, in the case of reconstruction, the pavement engineer can specify different values for the new asphalt concrete and aggregate base thicknesses and then select the design that yields a desirable predicted average distress rating ($DR_{LC}$) or terminal distress rating, DR($2n$).

## Disclosure statement

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