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Prediction of asphaltic overlay thickness for rehabilitating flexible pavement using Empirical-Markovian approach

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ABSTRACT

This paper presents a simplified Empirical-Markovian approach for estimating the asphaltic overlay thickness used in rehabilitating flexible pavement at the project level. The main element of the proposed Empirical-Markovian approach is the prediction of the future pavement conditions which are estimated using the heterogeneous discrete-time Markov model. The required heterogeneous transition probabilities are predicted for an original pavement structure by mainly relying on the first-year transition probabilities. The empirical overlay design model is developed assuming both original and overlaid pavements will exhibit similar performance trends over time as represented by the corresponding performances curves. This has been accomplished by requiring the deterioration transition probabilities associated with both pavements to be equal in values. The overlay model developed is mainly a function of the annual traffic growth rate, rehabilitation scheduling time, initial structural capacity associated with original pavement, and two calibration constants. The two calibration constants are the same ones obtained from the development of the empirical model used to estimate the heterogeneous transition probabilities associated with original pavement. A sample problem is presented to illustrate the use of the proposed overlay design model. The sample overlay design thicknesses predicted for variable rehabilitation scheduling time appear to be reasonable and consistent with the general practice.

Keywords: Heterogeneous Markov chain, Overlay design, Pavement performance, Pavement rehabilitation, Pavement management

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 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 works. However, rehabilitation works not only provide major enhancement of the pavement structure but it can substantially extend its service life. Pavement rehabilitation strategies typically include plain overlay, cold milling and overlay, and reconstruction. The placement of an asphalt overlay is commonly used by many highway agencies to rehabilitate existing flexible and rigid pavements (Zhou et al. 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). 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 increased traffic load applications. The most popular M-E method is the one that uses nondestructive testing (NDT) 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 (FWD). The overlay design thickness is then computed based on back-calculation of the multilayer linear elastic system (AI 1996, Mallela et al. 2008, Tutumluer & Sarker 2015). The pavement remaining strength can also be computed using the modified layer coefficients (AASHTO 1993, Tutumluer & Sarker 2015). 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 et al. 2015).

Pavement performance is not only a key pavement design parameter but it is 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) or distress rating (DR) which can be predicted using stochastic modeling. In particular, Markovian processes have been extensively used to predict pavement service condition over time for pavement management applications (Hong and Wang 2003, Abaza and Murad 2009, Lethanh and Adey 2013, Meidani and Ghanem 2015, Abaza 2015). Therefore, it is proposed to derive an empirical Markovian-based model to estimate the overlay design thickness under the assumption that both original and overlaid pavements will exhibit similar deterioration trends as represented by the corresponding performance curves. The performance curves can be developed using the discrete-time Markov model with the deterioration transition probabilities (i.e. deterioration rates) representing the main input parameters (Abaza 2015). In essence, the stated assumption can be satisfied by requiring the deterioration transition probabilities associated with both pavements to be equal. Of course, this would be a good practice provided the performance of the original pavement is a satisfactory 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.

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 heterogeneous 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, $\mathbf{P}(\mathbf{k})$. The state probabilities represent the proportions of pavement that are expected to exit in the various deployed pavement condition states at a specified future time. The sum of the state probabilities must add up to one as indicated by Equation (1).

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

where: $S^{(n)} = (S_1^{(n)}, S_2^{(n)}, S_3^{(n)}, \dots, S_m^{(n)})$

$$S^{(0)} = (S_1^{(0)}, S_2^{(0)}, S_3^{(0)}, \dots, S_m^{(0)})$$

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

The transition probability matrix is (mxm) 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 (2015) 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 maintenance and rehabilitation (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.

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

where:

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

$$0 \leq P(k)_{i,i} \leq 1.0, \quad 0 \leq P(k)_{i,i+1} \leq 1.0$$

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 (2015) reported that (10x10) 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 pavement condition index (PCI), distress rating (DR), or present serviceability index (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 estimated using Equation (3) as a function of the state mean distress ratings (B_i) and state probabilities, $S_i^{(k)}$, associated with the k th transition. The state mean distress ratings 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 distress rating with higher ratings indicating better pavement. The state probabilities can be estimated provided that the relevant transition probability matrices are available over an analysis period of (n) transitions.

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

where:

$$S^{(k)} = \begin{cases} S_1^{(k)}, & 90 < DR_1 \leq 100, & B_1 = 95 \\ S_2^{(k)}, & 80 < DR_2 \leq 90, & B_2 = 85 \\ S_3^{(k)}, & 70 < DR_2 \leq 80, & B_3 = 75 \\ \vdots & \vdots & \vdots \\ S_{10}^{(k)}, & 0 \leq DR_{10} \leq 10, & B_{10} = 5 \end{cases}$$

Once the distress ratings are estimated 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 provides 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 2015).

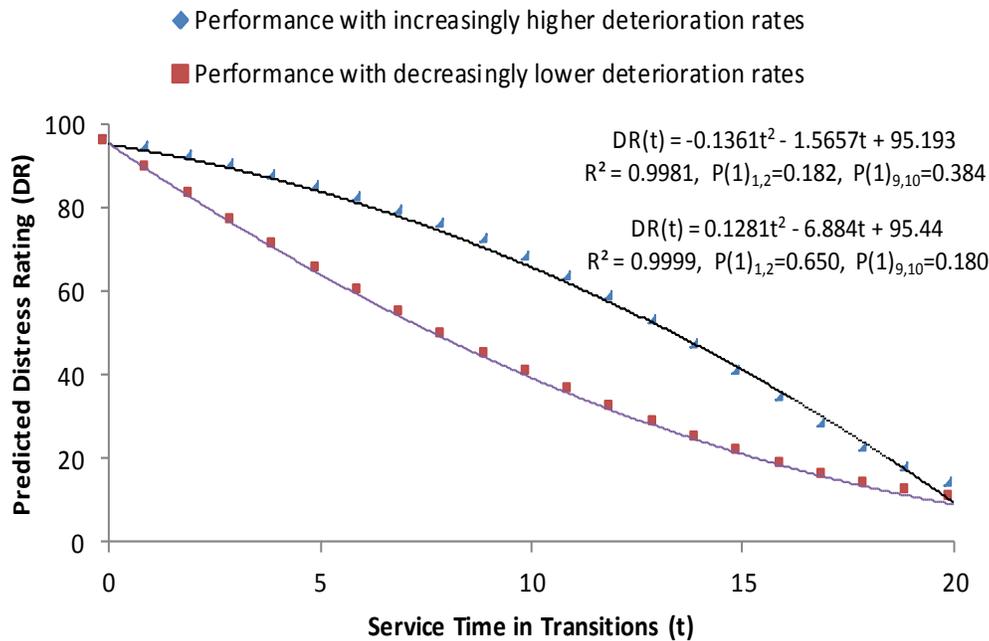


FIGURE 1 Sample pavement performance curves for original pavement predicted using heterogeneous Markov chain.

METHODOLOGY

The main drawback of using the heterogeneous 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 (2015) proposed an empirical model to predict the future heterogeneous 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 & B) can be estimated from the calibration procedure which relies on the minimization of the sum of squared errors (SSE) as outlined in Abaza (2015).

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

An empirical model similar to the one presented in Equation (4) is proposed to predict the future heterogeneous 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). 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 under the assumption of similar performances of both overlaid and original pavements. This assumption can be achieved by requiring the two sets of deterioration transition probabilities [$P(t+k)_{i,i+1}$ & $P(k)_{i,i+1}$] associated with both overlaid and original pavements, respectively, to be equal in values. This assumption implies that the performance curve associated with the overlaid pavement will resemble that of the original pavement as shown in Figure 2 with the same service life of (n) transitions.

The main factors used in Equation (5) are the traffic load and pavement strength factors raised to the powers (A & B), respectively. The load factor is a ratio between the 80kN 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, $S(k-1)$, to the corresponding value associated with the overlaid pavement, $S(t+k-1)$, 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 higher deterioration transition probabilities over time. Figure 2 shows (t) to be the overlay scheduling time in transitions, which is the same as the age associated with original pavement.

$$P(t+k)_{i,i+1} = P(k)_{i,i+1} \left(\frac{\Delta W(t+k)}{\Delta W(k)} \right)^A \left(\frac{S(k-1)}{S(t+k-1)} \right)^B \quad (k = 1, 2, \dots, t) \quad (5)$$

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 (6) as obtained from the simplification of Equation (5). This will ensure the performance curves associated with both pavements to be similar as shown in Figure 2. According to Equation (6), the strength required for overlaid pavement at the (t+k-1)th transition is a function of the strength 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 & B) are assumed to be the same as the ones used in Equation (4) which is applicable to original pavement. This is a reasonable assumption since both pavements are expected to exhibit similar performances as shown in Figure 2.

$$S(t+k-1) = S(k-1) \left(\frac{\Delta W(t+k)}{\Delta W(k)} \right)^{A/B} \quad (k = 1, 2, \dots, t) \quad (6)$$

where: $\Delta W(t+k) = W(t+k) - W(t+k-1)$

$$\Delta W(k) = W(k) - W(k-1)$$

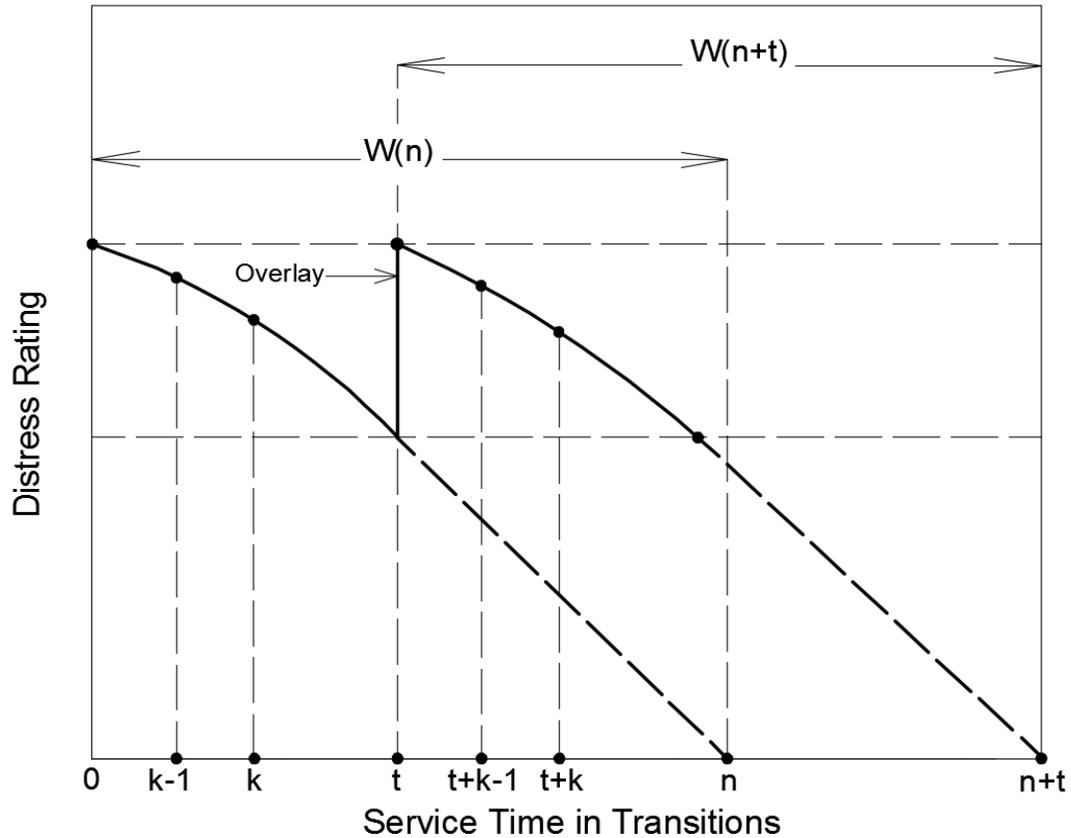


FIGURE 2 Typical overlay plan with similar performance curves for both original and overlaid pavements.

The accumulated traffic load applications at the k th transition, $W(k)$, in terms of the 80kN single axle load applications, can be obtained from multiplying the first-year load applications (W_f) by the corresponding traffic growth factor, $GF(k)$, as defined in Equation (7). The deployed formula for estimating the traffic growth factor is the one proposed by the Asphalt Institute and it is a function of the uniform annual traffic growth rate (r) (AI 1999).

$$W(k) = W_f \times GF(k) = W_f \left(\frac{(1+r)^k - 1}{r} \right) \quad (7)$$

The ratio associated with the traffic load factor can then be simplified as presented in Equation (8). The outcome of this simplification indicates that this ratio is only dependent on the traffic growth rate (r) and the overlay scheduling time (t) in transitions. Each transition is typically assumed to be equal to one year.

$$\frac{\Delta W(t+k)}{\Delta W(k)} = \frac{W_f \times GF(t+k) - W_f \times GF(t+k-1)}{W_f \times GF(k) - W_f \times GF(k-1)} = (1+r)^t \quad (8)$$

The derived term for the ratio associated with the traffic load factor is then substituted in Equation (6) to yield Equation (9). However, Equation (9) is derived at the 1st transition ($k=1$) implying that the required strength for estimating the overlay design thickness at time (t) is dependent on the initial strength associated with the original pavement, $S(0)$. This is a reasonable requirement since the strength for pavement design is typically estimated at the beginning of the pavement service life.

$$S(t) = S(0) (1+r)^{tA/B} \quad (9)$$

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 (8). 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.

$$\frac{W(n+t)}{W(n)} = \frac{W_f \times GF(n+t) - W_f \times GF(t)}{W_f \times GF(n)} = (1+r)^t \quad (10)$$

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 in its manual for design of flexible pavement (Caltrans 2008). Therefore, it is proposed to replace the structural capacity used in Equation (9) by the structural number to yield Equation (11). However, the structural number associated with the asphalt concrete layer, SN_1 , is used in Equation (11), which 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 2015).

$$SN_1(t) = SN_1(0) (1+r)^{tA/B} \quad (11)$$

According to Equation (11), the design structural number, $SN_1(t)$, for the asphalt concrete layer associated with the overlaid pavement is to be estimated based on the corresponding value associated with the original pavement, $SN_1(0)$, annual traffic growth rate (r), overlay scheduling time (t), and the two calibration exponents (A & B). However, the structural number associated with the overlay design thickness, $SN_o(t)$, has to be estimated using Equation (12) to account for the remaining structural capacity associated with the existing asphalt concrete layer, $SN_1(0)$. A remaining strength parameter, $R(t)$, is therefore introduced in Equation (12) to allow for reducing the structural capacity of existing asphalt concrete layer.

$$SN_o(t) = SN_1(t) - R(t) SN_1(0) \quad (12)$$

The required overlay design thickness can then be estimated using Equation (13). The structural number associated with the overlay design thickness is divided by the relative strength coefficient (a_o) to yield the overlay design thickness, $D(t)$, in centimeter. The relative strength layer coefficients have been used by AASHTO empirical design method to convert design structural numbers into equivalent design thicknesses (AASHTO 1993). The layer coefficients have been correlated to key compressive strength parameters such as the California bearing capacity (CBR), resilient modulus and Marshall stability.

$$D(t) = 2.5 \left(\frac{SN_o(t)}{a_o} \right) \quad (13)$$

The remaining strength parameter, $R(t)$, can be estimated from both the destructive and non-destructive testing of pavement (Huang 2004). Abaza (2015) 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 maximum DR value, DR_{max} , as presented in Equation (14).

$$R(t) = \frac{DR(t)}{DR_{max}} \quad (14)$$

The required $DR(t)$ can be the predicted value as obtained from Equation (3). Generally, there are two types of pavement performance as depicted in Figure 1 (Abaza and Murad 2009). The first type is associated with increasingly higher deterioration transition probabilities with the corresponding model indicated by Equation (15a). While the second one is associated with decreasingly lower deterioration transition probabilities as defined in Equation (15b). The maximum DR value is simply the last term in these two models.

$$DR(t) = -at^2 - bt + c \quad (15a)$$

where: $P_{1,2} < P_{2,3} < P_{3,4}, \dots < P_{m-1,m}$

$$DR(t) = at^2 - bt + c \quad (15b)$$

where: $P_{1,2} > P_{2,3} > P_{3,4}, \dots > P_{m-1,m}$

It is expected that the remaining strength parameter, $R(t)$, as estimated from Equation (14) will provide a reasonable estimate of the pavement remaining strength associated with the asphalt concrete layer. This is especially true because the distress rating is essentially a quality measure of the asphalt concrete layer as obtained from the field assessment of prevailing pavement distresses.

Resurfacing of flexible pavement is occasionally preceded by cold milling of a certain thickness of the existing asphalt concrete layer. Cold milling is typically performed when the overall condition of the asphalt concrete surface is not stable enough to adequately support a plain overlay. Therefore, the overlay design thickness can be adjusted by multiplying the cold milling thickness, D_m , by the remaining strength parameter to yield the adjusted overlay design thickness, $D'(t)$, as presented in Equation (16).

$$D'(t) = D(t) \cdot R(t) \cdot D_m \quad (16)$$

SAMPLE PRESENTATION

The proposed Empirical-Markovian approach has been used to estimate the overlay design thickness for 4-lane urban arterial located in the city of Nablus, West Bank, Palestine. The arterial is paved with flexible pavement comprised of 12 cm, $SN_1(0)=2.1$, high-stability, hot-mix asphalt concrete surface on top of 50 cm aggregate base. This pavement structure was designed to support 5-million design ESAL, $W(n)$, over an analysis period (n) comprised of 20 years (i.e. 20 transitions). The associated annual traffic growth rate (r) is 4% and the relative strength coefficient for high-stability asphalt concrete (a_0) is 0.44.

Abaza (2015) predicted the performance of this arterial using the heterogeneous deterioration transition probabilities obtained from the empirical model presented in Equation (4). Two types of pavement performance were identified for this arterial as outlined earlier and depicted in Figure 1. The superior performance has prevailed over the vast majority of the arterial pavement that was built on subgrade with good bearing capacity, while inferior performance is spotted over few pavement sections that were constructed on poor subgrade. However, the original pavement structure was designed based on good bearing capacity as it was the predominant case. The corresponding performance curves shown in Figure 1 mainly provide a plot of the project predicted distress rating, $DR(t)$, versus service time in transitions (t). Their best-fit models are indicated by Equations (17a) & (17b) for performances with increasingly higher and decreasingly lower deterioration transition probabilities, respectively.

$$DR(t) = -0.1361t^2 - 1.5657t + 95.193, \quad DR_{max}=95.193 \quad (17a)$$

$$DR(t) = 0.1281t^2 - 6.884t + 95.44, \quad DR_{max}=95.44 \quad (17b)$$

Equation (14) has been used to estimate the remaining strength parameter, $R(t)$, with the required distress ratings, $DR(k)$, are predicted using Equations (17a) & (17b). Tables 1 and 2 provide the corresponding $R(t)$ values as a function of the overlay scheduling time (t) for pavement performances with increasingly higher and decreasingly lower deterioration rates, 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 overlay scheduling time (t) increases. The overlay scheduling time has been selected to vary 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.

TABLE 1 Sample Overlay Design Thickness for Pavement Performance with Increasingly Higher Deterioration Transition Probabilities (Superior Performance)

t (yrs.)	$W(n+t) \times 10^6$	DR(t)	R(t)	$SN_1(t)$	$SN_0(t)$	D(t), cm
5	6.08	83.96	0.882	2.64	0.79	4.49
6	6.33	80.90	0.850	2.76	0.98	5.57
7	6.58	77.56	0.815	2.89	1.18	6.70
8	6.84	73.96	0.777	3.03	1.40	7.95
9	7.12	70.08	0.736	3.17	1.62	9.20
10	7.40	65.93	0.693	3.32	1.86	10.57

TABLE 2 Sample Overlay Design Thickness for Pavement Performance with Decreasingly Lower Deterioration Transition Probabilities (Inferior Performance)

t (yrs.)	$W(n+t) \times 10^6$	DR(t)	R(t)	$SN_1(t)$	$SN_0(t)$	D(t), cm
5	6.08	64.22	0.673	2.96	1.55	8.81
6	6.33	58.75	0.616	3.17	1.88	10.68
7	6.58	53.53	0.561	3.39	2.21	12.56
8	6.84	48.57	0.509	3.64	2.57	14.60
9	7.12	43.86	0.460	3.89	2.92	16.59
10	7.40	39.41	0.413	4.17	3.30	18.75

Tables 1 & 2 indicate that as the pavement service time (t) increases from 5 to 10 years, the remaining strength parameter, R(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 structural number associated with the overlaid pavement, $SN_1(t)$, is estimated using Equation (11) with $SN_1(0)$ being equal to 2.1 as outlined earlier. The model exponents (A & B) have been assigned the values of (1.4 & 1.2) for superior performance and (0.7 & 0.4) for inferior performance as reported by Abaza (2015) based on the calibration of the empirical model presented in Equation (4) and used to derive the sample performance curves shown in Figure 1.

The structural number associated with the overlay design thickness, $SN_O(t)$, is computed from Equation (12) using 0.44 relative strength coefficient (a_o). The overlay design thickness, $D(t)$, is then determined from Equation (13) with the corresponding results provided in Tables 1 & 2 for both types of pavement performance. It can be noted that the overlay design thickness seems to be appropriate in the case of superior performance. However, the overlay design thickness is about 80-90% higher in the case of inferior performance compared to superior 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 corresponding subgrade bearing capacity. Plain overlay is typically applied at early pavement service times; however, cold milling is required at advanced service times as the pavement surface is not sound enough to support a plain overlay. For example, 5 cm cold milling thickness (D_m) applied at 10-year service time results in 7.1 cm adjusted overlay design thickness, $D'(t)$, computed using Equation (16) in the case of superior performance. The outcome of this example is in compliance with the general policy adopted by the Department of Public Works, City of Nablus, for rehabilitating the major arterials within its jurisdiction once they reach 10 years of service time, namely 5-cm cold milling followed by 7-cm high stability asphalt concrete overlay.

CONCLUSION

Based on the sample overlay design thicknesses presented for an urban arterial, the author believes the proposed Empirical-Markovian model provides a potential approach for asphalt overlay design. The main requirement for developing a distinct overlay design model at the project level is the derivation of the model exponents (A & B) since the other needed parameters are readily available. The model exponents are assumed to be the same as those deployed by the model used to predict the heterogeneous transition probabilities for original pavement (i.e. Equation (4)), which is a valid assumption provided that the performance curves of both original and overlaid pavements are similar. The performance curves are expected to be similar because the corresponding deterioration transition probabilities are set to be the same. The model exponents can be estimated from the minimization of sum of squared errors (SSE), wherein the error is defined as the difference between the annual predicted and observed distress ratings.

Abaza (2015) outlined the procedure to calibrate the model presented in Equation (4) for the purpose of obtaining reliable estimates of the model exponents (A & B). The main requirement is to carry out annual assessment of pavement distress as typically needed for pavement management applications. The collected distress data can then be converted to equivalent annual observed distress ratings using an appropriate formula. The corresponding annual predicted distress ratings are estimated from the outlined heterogeneous Markov model with the relevant heterogeneous deterioration transition probabilities derived from the empirical model indicated by Equation (4). A simplified trial and error approach was outlined by Abaza (2015) to perform the SSE procedure that would lead to reliable estimates of the model exponents at the project level. However, it is recommended that local highway agencies interested in using the proposed overlay design approach to develop a distinct model for each set of pavement projects with similar material properties and traffic conditions.

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