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Simplified novel approach for estimating HMA overlay thickness schedule using long-term performance indicators

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ABSTRACT

A simplified novel approach for estimating the HMA overlay thickness is proposed in this paper for flexible pavement resurfacing schedules. The overlay thickness is mainly estimated from the difference between the existing asphaltic thickness and reduced asphaltic thickness. The reduced asphaltic thickness is estimated as the sum of two components: a bottom component unaffected by pavement distresses and reduced thickness of a top component affected by pavement distresses. The reduced thickness of the top component is estimated from multiplying its thickness after subtracting the cold milling thickness by an asphaltic remaining strength factor. The remaining strength factor is defined as a ratio of average distress rating (DR) associated with the pavement remaining service life and average DR associated with the entire service life. This ratio is raised to the power (K) to be estimated from the minimisation of sum of squared errors. The two required DR averages are estimated from the performance curve for a particular pavement project. The remaining strength factor can also be defined as a ratio of the average surface deflection ($\bar{D}_{80}$) and tolerable deflection at the surface (TDS) as deployed by the Caltrans mechanistic-empirical design approach. The reduced asphaltic thickness is mainly used to estimate the overlay thickness required at a specified service time. The two case studies presented have indicated the effectiveness of the proposed approach in estimating overlay thickness schedules with relatively low errors.

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Flexible pavement; overlay design; pavement performance; pavement rehabilitation; pavement management

1. Introduction

Pavement management at the project level requires a simplified and reliable approach to identify the appropriate rehabilitation plan at a specified pavement service time. Potential rehabilitation plans for flexible pavement typically include plain overlay, cold milling and overlay, and major reconstruction. Life-cycle cost–benefit analyses have indicated that major reconstruction is economically unfeasible compared to the other potential rehabilitation plans (Santos and Ferreira 2013, Heravi and Esmaeeli 2014, Hong and Prozzi 2015). Therefore, practitioners place more emphasis on plain overlay, and cold milling and overlay. In practice, there are several methods that can estimate the required overlay thickness associated with a particular pavement project mainly replying on the present pavement distress assessment or surface deflection measurements. However, there is a vital need to develop a simplified and effective approach that can estimate the required overlay thickness schedule as a function of pavement service time. The required approach shall take into consideration the pavement long-term performance which is a key requirement for effective pavement management practices.

The general approach deployed for estimating the reduced (i.e. effective) asphaltic strength and consequently the required overlay thickness mainly attempts to compensate the asphaltic surface for the strength loss it suffered over its service time. Presently, there are two general approaches used by practitioners to estimate the reduced asphaltic strength. The first approach is a mechanistic-based one that mostly relies on surface deflection measurements obtained using non-destructive testing procedures such as the Falling-Weight Deflectometer (FWD) procedure (Zhou *et al.* 1992, Hoffman 2003, Sarker *et al.* 2015, Tutumluer and Sarker 2015, Nam *et al.* 2016, Smith *et al.* 2017). The deflection measurements are then used in what is known as back-calculation of the multi-layered linear elastic theory to yield the reduced asphaltic strength in terms of reduced modulus. The second approach is an empirical one known as the effective thickness approach or component analysis method which attempts to estimate the reduced asphaltic strength using equivalency conversion factors/correction factors assigned based on the outcome of pavement distress surveys (AASHTO 1993, AI 1996, Huang 2004, Abaza 2018, Bianchini *et al.* 2018).

Recent developments in overlay design have applied modified approaches including mechanistic-empirical ones. For example, Zhou *et al.* (2010) presented a comprehensive mechanistic-empirical system to design a balanced asphaltic overlay based on traffic loadings, climate conditions, existing pavement conditions, and material properties of asphalt overlay mix. Maji *et al.* (2016) developed a comprehensive probabilistic approach for asphaltic overlay design that can accommodate variations in the design parameters which include layer thicknesses, layer moduli, vehicle damage factor, lane distribution factor, and traffic growth rate. Nobakht *et al.* (2017) proposed a rehabilitation strategy based on structural capacity, which mainly applies a damage ratio estimated

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from the FWD data and a rutting index estimated from distress assessment. Le *et al.* (2017) developed a simple regression model for estimating overlay design thickness based on the mechanistic-empirical approach. The model is a function of the layer thicknesses, asphaltic modulus ratio, subgrade condition and traffic volume. Bianchini *et al.* (2018) proposed a modified overlay procedure that accounts for the structural condition of the existing asphaltic surface mainly using an asphaltic correction factor estimated from the existing load-related distresses. Abaza (2018) proposed an Empirical-Markovian model to predict the overlay design thickness as a function of the original structural capacity and other related parameters including pavement deterioration transition probabilities.

The previously outlined overlay design approaches mainly focus on estimating the required overlay thickness depending on the current roadway conditions, thus making them inappropriate for pavement management applications. The main objective of this paper is to develop a simple but yet effective approach that can estimate the reduced asphaltic thickness schedule as a function of pavement long-term performance at the project level. The long-term performance is to be defined as a function of pavement distress rating predicted using an Empirical-Markovian approach. The proposed approach can estimate the overlay thickness schedule required for the two potential rehabilitation plans, namely plain overlay, and cold milling and overlay. The proposed approach applies an asphaltic remaining strength factor that can be estimated from either the predicted long-term performance indicators or surface deflection measurements. The long-term performance indicators can be deployed by local governments mainly replying on pavement distress assessment while deflection measurements are typically performed by state highway agencies. The long-term performance indicators are to be estimated from pavement performance curves generated using either deterministic or probabilistic prediction approaches (Lethanh & Adey 2013, Amin 2015, Abaza 2018, Abed *et al.* 2019, Fuentes *et al.* 2019). The asphaltic remaining strength factor can also be estimated as a ratio of the average surface deflection and tolerable deflection at the surface as deployed by the Caltrans mechanistic-empirical design approach.

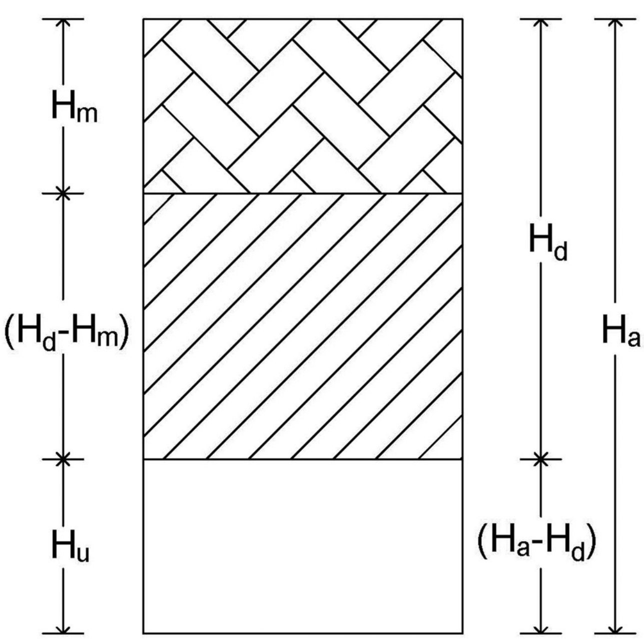


Figure 1. Basic thickness components of the asphaltic surface used in estimating the reduced asphaltic thickness.

2. Methodology

This section presents the main approach for estimating the reduced (i.e. effective) asphaltic surface thickness to be used in computing the required hot-mix asphalt (HMA) overlay thickness. It is assumed that the asphaltic surface layer is the main layer that suffers major strength loss over time. Other underlying pavement layers are assumed to endure minor strength losses which can be neglected when determining the corresponding overlay thickness. The methodology section presents two approaches to calibrate and estimate the asphaltic remaining strength factor used to compute the reduced asphaltic thickness. The first approach relies on the pavement long-term performance defined in terms of the annual distress rating (DR), while the second one deploys pavement surface deflections.

2.1. Estimation of reduced asphaltic thickness

Estimation of the reduced asphaltic surface thickness (H''_a) is mainly dependent on the four thickness components shown in Figure 1, namely existing asphaltic thickness (H_a) as measured in the field, cold milling thickness (H_m) specified according to the pavement distress condition, distressed asphaltic thickness (H_d) before cold milling, and asphaltic thickness unaffected by distresses (H_u) called distress-free thickness. As a simplification, the following short names will be used for the various thickness components:

H_a = existing asphaltic thickness, H''_a = reduced asphaltic thickness (H''_a < Ha), H_m = cold milling thickness, H_d = distressed thickness, H''_d= reduced distressed thickness ($H''_d < H_d$), H_u = distress-free thickness.

The reduced asphaltic thickness (H''_a) represents the thickness of new HMA that is equivalent to the existing asphaltic thickness (H_a). The reduced asphaltic thickness (H''_a) is defined as the sum of two parts as defined in Equation (1): the first part is the distress-free thickness (H_u) assumed to suffer no loss in strength, and the second part is the reduced distressed thickness (H''_d) assumed to have suffered strength loss.

$$H''_a = H_u + H''_d \tag{1}$$

The reduced distressed thickness (H''_d) is estimated as the difference between the distressed thickness (H_d) and cold milling thickness (H_m) with the difference multiplied by an asphaltic remaining strength factor, $F_s(t)$, as indicated by Equation (2). The asphaltic remaining strength factor is to be estimated based on available records/predicted indicators of pavement distress condition as later explained. Equation (2) is applicable to the two most popular rehabilitation plans associated with flexible pavement, namely plain overlay

($H_m = 0$), and cold milling and overlay ($H_m > 0$).

$$H''_d = F_s(t) \times (H_d - H_m) \quad , H_m \leq H_d \tag{2}$$

The distressed thickness (H_d) is to be estimated from multiplying the existing asphaltic thickness (H_a) by a vertical distress spread ratio (λ) as provided by Equation (3a). The distress-free thickness (H_u) can similarly be estimated using Equation (3b). The vertical distress spread ratio (λ) can be estimated from the testing of asphaltic core samples. It can also be estimated based on professional experience and engineering judgement. It simply measures how deep the surface distresses are propagating into the asphaltic layer. Therefore, it is expected to reach one for thin asphaltic surfaces, but it can be less than one for thick asphaltic surfaces.

$$H_d = \lambda \times H_a, \quad \lambda \leq 1.0 \tag{3a}$$

$$H_u = (1 - \lambda) \times H_a \tag{3b}$$

where $H_a = H_d + H_u$

Equation (1) can then be redefined as indicated by Equation (4) which incorporates the value of (H''_d) as obtained from Equation (2) and the values of (H_d) and (H_u) as presented in Equation (3). Therefore, the reduced asphaltic thickness (H''_a) can be estimated from Equation (4) as a function of the existing asphaltic thickness (H_a), cold milling thickness (H_m), vertical distress spread ratio (λ), and asphaltic remaining strength factor, $F_s(t)$. The reduced asphaltic thickness (H''_a) is expected to be lower than the existing asphaltic thickness (H_a) with their difference typically becomes larger as the pavement distress condition gets worse over time, consequently the required overlay thickness becomes larger. All parameters involved in Equation (4) can vary over time with the exception of the existing asphaltic thickness (H_a).

$$H''_a(t) = (1 - \lambda) \times H_a + F_s(t) \times (\lambda \times H_a - H_m) \quad , F_s(t) \leq 1.0 \tag{4}$$

The asphaltic remaining strength factor can be estimated using the present values associated with key performance indicators such as the present serviceability index (PSI) and international roughness index (IRI) (Abaza 2019). In this paper, it is proposed to estimate it using long-term performance indicators derived as a function of pavement distress rating (DR) to be later defined. Equation (5) is proposed to estimate the asphaltic remaining strength factor as a ratio of the average DR associated with the remaining service life (T −t) and average DR over the entire service life (T) with the ratio raised to power (K) to be estimated from the calibration procedure outlined later. Capital (T) represents the pavement service life and small (t) denotes the rehabilitation scheduling time (or service time), in years, as shown in Figure 2. The $\overline{DR}(T - t)$ value represents the average DR over the remaining service life (T−t), and $\overline{DR}(T)$ value designates the average DR over the entire service life (T). Both used DR averages represent measures of the pavement long-term performance as depicted in Figure 2.

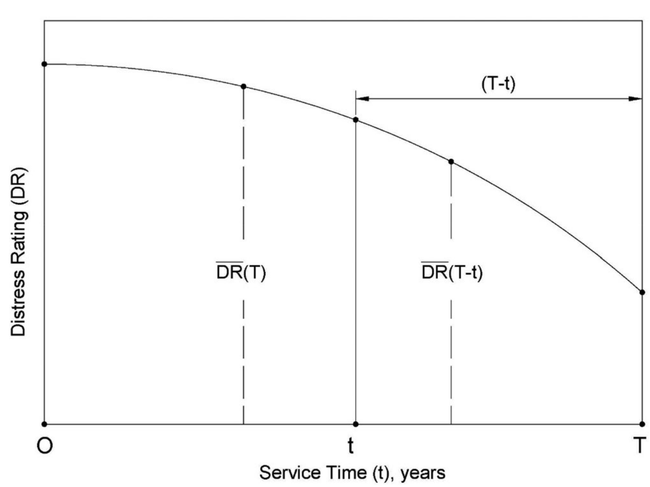


Figure 2. Typical pavement long-term performance curve with average distress ratings used in estimating the asphaltic remaining strength factor.

$$F_s(t) = \left(\frac{\overline{DR}(T - t)}{\overline{DR}(T)}\right)^K, \quad \overline{DR}(T - t) < \overline{DR}(T) \tag{5}$$

where

$$\overline{DR}(T - t) = \frac{\sum_{j=t}^{T} DR(j)}{T - t + 1}$$

$$\overline{DR}(T) = \frac{\sum_{j=0}^{T} DR(j)}{T + 1}$$

Therefore, estimation of the asphaltic remaining strength factor as defined by Equation (5) requires predicting two average distress ratings as indicators of the pavement long-term performance. The current practice is to estimate the overlay thickness for a particular pavement project mainly depending on its current pavement condition without any consideration to its remaining long-term performance. Pavement long-term performance curves similar to the one shown in Figure 2 can be generated using, for example, an Empirical-Markovian approach as outlined in a subsequent section (Abaza 2018). However, there are other deterministic and probabilistic approaches that can be used to develop similar pavement performance curves.

The required overlay thickness, $h_e(t)$, can be estimated from the difference of the existing asphaltic surface thickness (H_a) and the corresponding reduced asphaltic thickness, $H''_a(t)$, as indicated by Equation (6). This is essentially an attempt to compensate the asphaltic surface for the loss in structural capacity it endured over a service time of (t) years. The other underlying pavement layers generally suffer minor strength losses that are typically ignored (Abaza 2018). Equation (6) also applies a load factor, $F_L(t)$, which accounts for the impact of anticipated future growth in traffic loads. Abaza (2019) indicated that the load factor is generally in the range of (1.10–1.30) for an annual traffic growth rate of (3–6%), respectively. The load factor as applied to Equation (6) linearly adjusts the overlay thickness to reflect the impact of future increases in traffic load applications.

$$h_e(t) = [H_a - H''_a(t)] \times F_L(t) \quad H''_a(t) < H_a \tag{6}$$

The use of the previously outlined approach for estimating the reduced asphaltic thickness as a function of the asphaltic remaining strength factor, $F_s(t)$, is mainly dependent on two main parameters, namely the long-term performance in terms of annual distress ratings predicted over a service life of (T) years, and the constant power (K) used in Equation (5). The constant power (K) can effectively be estimated using the minimisation of sum of squared errors (SSE) as defined in Equation (7). In this case, the error is defined as the difference between the overlay thickness, $h_e(t)$, estimated using the previously outlined procedure and observed overlay thickness, $h_o(t)$, as would actually be provided using a reliable overlay design procedure including the 'prescription' procedure typically deployed by local governments.

$$\text{Minimize:SSE} = \sum_{j=t}^{T} [h_e(j) - h_o(j)\ ]^2 \qquad (7)$$

The minimisation procedure as indicated by Equation (7) is to be performed at the project level or project group considering a remaining service life consisting of (T−t+1) years over which the annual estimated and observed overlay thicknesses are required. Therefore, Equation (7) requires an overlay thickness schedule to be estimated over an analysis period of (T−t+1) years.

2.2. Prediction of pavement long-term performance

The pavement long-term performance of a particular project can be estimated using DR as an indicator of pavement condition. Equation (8) can be used to estimate the annual average distress rating, DR(k), as a product sum of the state average distress ratings, $\overline{DR}_i$, and corresponding state probabilities, $S_i^{(k)}$, associated with the kth transition (i.e. one year) for a particular pavement project. The distress rating is to be estimated on a scale of 100 points with higher ratings indicating better pavements. The state average distress rating is defined as the average of the lower and upper DR ratings (LDR_i & UDR_i) used to define the *i*th condition state as indicated by Equation (8). Therefore, the state average distress ratings take on the values of (5, 15, 25, … ., 95) when 10 condition states (m) are deployed with equal 10-point DR range defining each state.

$$DR(k) = \sum_{i=1}^{m} \overline{DR}_i \times S_i^{(k)} \quad (k = 1, 2, \ldots., m) \qquad (8)$$

where $\overline{DR}_i = (LDR_i + UDR_i)/2$

The state probabilities used in Equation (8) represent the pavement proportions that exist in the various deployed condition states at any given transition (i.e. year). They can either be estimated from the periodical assessment of pavement distresses (i.e. defects) or predicted using the Markov model. Several versions of the Markov model have been used to predict the pavement long-term performance (Lethanh and Adey 2013, Amin 2015, Lethanh *et al.* 2015, Meidani and Ghanem 2015, Abaza 2018). Abaza (2018) used the non-homogenous discrete-time Markov model as defined in Equation (9) to predict the future state probabilities associated with a Markov chain of (m) condition states. This model can incorporate a different transition probability matrix (TPM) for each transition (i.e. time interval typically specified as one-year) considering an analysis period comprised of (n) transitions. The sum of state probabilities must add up to one at any given transition. For a new project, it is typical to assign all pavements to state (**1**), which means the 1st initial state probability, $S_1^{(0)}$, takes on the value of one with the others assigned zero values.

$$S^{(n)} = S^{(0)}\left(\prod_{k=1}^{n} P(k)\right) \qquad (9)$$

where $S^{(n)} = (S_1^{(n)}, S_2^{(n)}, S_3^{(n)}, \ldots., S_m^{(n)})$, $S^{(0)} = (S_1^{(0)}, S_2^{(0)}, S_3^{(0)}, \ldots., S_m^{(0)}) = (1, 0, 0, 0, \ldots.., 0)$ for new pavement, $\sum_{i=1}^{m} S_i^{(k)} = 1.0$

An example of potential transition probability matrix (TPM) is presented in Equation (10) wherein only two transitions are allowed for pavement deterioration (Abaza 2018). This means at the end of the *k*th time interval, the pavement can either stay in the current condition state (**i**) with a probability of $P(k)_{i,i}$ or exit to the next worst state (**i+1**) with a probability of $P(k)_{i,i+i}$. The transition probabilities can be different for each time interval as they represent the pavement deterioration rates, which they typically get higher in values over time because of the progressive increase in traffic loading and progressive decrease in pavement structural capacity. The TPM indicated by Equation (10) only represents pavement deterioration, but it can also incorporate below the main diagonal the improvement rates resulting from pavement maintenance and rehabilitation. The sum of any row in the TPM must add up to one.

$$P(k) = \begin{pmatrix} P(k)_{1,1} & P(k)_{1,2} & 0 & 0 & 0 & \cdots & 0 \\ 0 & P(k)_{2,2} & P(k)_{2,3} & 0 & 0 & \cdots & 0 \\ 0 & 0 & P(k)_{3,3} & P(k)_{3,4} & 0 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & 0 & \cdots & P(k)_{m-1,m-1} & P(k)_{m-1,m} \\ 0 & 0 & 0 & 0 & 0 & \cdots & P(k)_{m,m} \end{pmatrix} \qquad (10)$$

Therefore, Equations (8)–(10) can be used to predict the pavement long-term performance as a function of the annual DR for a particular project over a specified service life with two examples shown in Figure 3. In these examples, the non-homogenous transition probabilities are estimated for a service life of 15 years (15 transitions) using an Empirical-Markovian model that accounts for the growth in traffic loading and degradation of the pavement strength over time (Abaza 2018). The main input data for the model include the initial and terminal transition probabilities [i.e. $P(1)_{1,1}$ & $P(1)_{9,10}$] for the 1st transition (m = 10). The remaining transition probabilities for each transition are estimated using linear interpolation. A case study is later presented to illustrate the use of the pavement long-term performance depicted in Figure 3 to calibrate and estimate the asphaltic remaining strength factor defined in Equation (5).

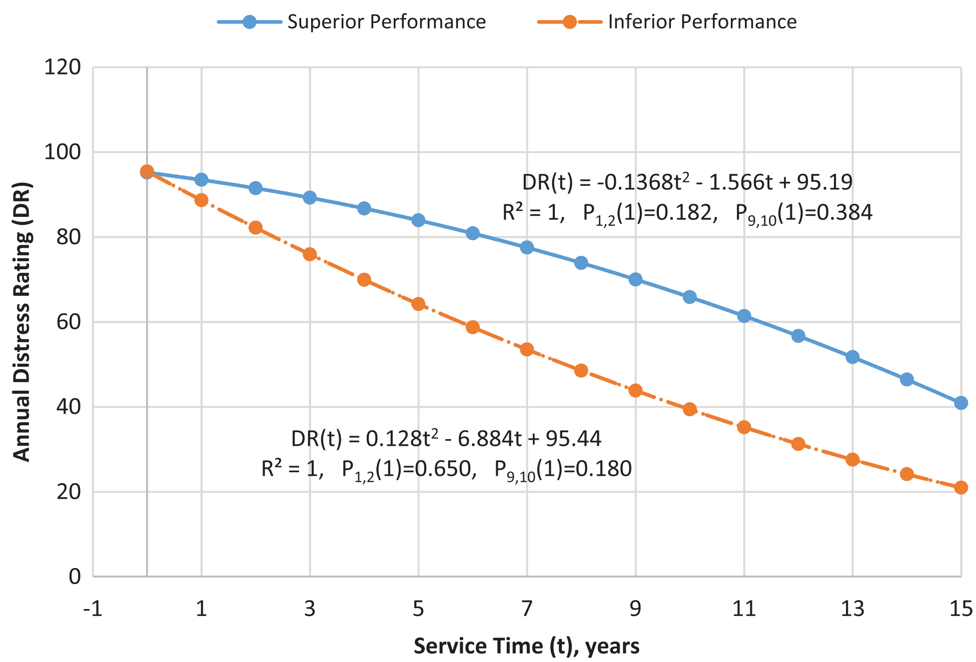


Figure 3. Sample pavement long-term performance curves predicted using the non-homogenous discrete-time Markov model.

2.3. Pavement distress assessment

Field assessment of pavement distresses (i.e. defects) is typically used to estimate the state and transition probabilities associated with a particular time interval (i.e. transition). The initial state probabilities required by the Markov model can be estimated from conducting one-cycle of pavement distress assessment, whereas two consecutive cycles are needed to estimate the present transition probabilities (Abaza 2016). A highway pavement project is typically divided into small lane segments with length in the range of 50–100 m. Each segment is then surveyed for the extent and severity of prevailing pavement defects with a corresponding DR is assigned on a scale of 0–100 points. Higher DR ratings indicate better pavements.

Abaza (2016) proposed simple models for estimating the observed DR for a lane segment mainly relying on the two most significant load-related pavement defects, namely cracking and deformation. Equation (11) presents an example of such models wherein DR is estimated for each lane segment using the localised cracked and deformed areas multiplied by their corresponding severity factors. The average DR value associated with all pavement segments is then computed to represent the distress condition of the entire highway project. Pavement distress assessment is typically conducted on annual or biennial basis.

$$DR = \left(\frac{3A_S - \sum_i SF_{C_i}A_{C_i} - \sum_i SF_{D_i}A_{D_i}}{3A_S}\right) \times 100 \qquad (11)$$

where $\sum_i SF_{C_i}A_{C_i} + \sum_i SF_{D_i}A_{D_i} \leq 3A_S$, $\sum_i A_{C_i} + \sum_i A_{D_i} \leq A_s$, A_S = entire surface area of the pavement lane segment (m^2), 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), 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).

In addition, Abaza (2016) provided the mathematical models to be used in estimating the transition probabilities associated with two consecutive cycles of pavement distress assessment typically separated by one-time interval of one-year length. However, these models are only applicable to the TPM presented in Equation (10).

2.4. Estimation of $F_s(t)$ by surface deflections

Estimation of the reduced asphaltic thickness (H''_a) as defined in Equation (4) mainly relies on the asphaltic remaining strength factor. The pavement long-term performance as a function of the annual distress rating has been used as the main input requirement to estimate the remaining strength factor for a particular pavement project. However, other pavement performance indicators can be used to estimate the remaining strength factor including PSI, IRI and surface deflections.

The mechanistic-empirical (ME) design approaches typically use surface deflections as a measure of pavement performance (AI 1999, AASHTO 2015, Caltrans 2017). Recent Caltrans design manual has presented a new ME design approach wherein a trial pavement structure is required to be analysed for fatigue cracking, rutting, and ride quality performance (Caltrans 2017). This typically requires a large number of computations using vast computers. Therefore, according to Caltrans, the ME approach is more of an analysis rather than a design procedure. However, both AASHTO and Caltrans use the IRI as a key performance indicator for analysing ride quality performance, and pavement surface deflections as a key performance indicator to analyse rutting (AASHTO 2015, Caltrans 2017).

In particular, the Caltrans ME design approach applies the 80th percentile deflection value (D_{80}) for a test section with a sample of surface deflection measurements as defined in Equation (12) wherein ($\bar{D}$) and (S_D) represent the mean and standard deviation associated with the surface deflection measurements, respectively.

$$D_{80} = \bar{D} + 0.84 \times S_D \tag{12}$$

For a particular rehabilitation project, the average D_{80} for all test sections ($\bar{D}_{80}$) is to be compared against the specified tolerable deflection at the surface (TDS). The TDS is specified based on traffic index (TI) and thickness of existing asphaltic layer (H_a) (Caltrans 2017). Consequently, the asphaltic remaining strength factor proposed in this paper can be defined as a ratio of TDS and $\bar{D}_{80}$ as indicated by Equation (13).

$$F_s(t) = \left(\frac{TDS}{\bar{D}_{80}(t)}\right)^K, \quad TDS \le \bar{D}_{80}(t) \tag{13}$$

The traffic index needed to estimate TDS is computed as a function of the original design load applications in terms of 80kN equivalent single axle load applications (W_{80}). According to Caltrans (2017), the TI is computed as a function of (W_{80}) using Equation (14).

$$TI = 9.0 \times \left(\frac{W_{80}}{10^6}\right)^{0.119} \tag{14}$$

Therefore, the asphaltic remaining strength factor can be calibrated and estimated using surface deflection measurements provided that highway agencies have access to the equipment needed to perform surface deflection testing such as the Falling-Weight Deflectometer. A case study is later presented to demonstrate the use of surface deflections in calibrating and estimating the asphaltic remaining strength factor defined by Equation (13).

2.5. Application to pavement management

Pavement rehabilitation management seeks to find optimal solutions for identifying and scheduling of potential pavement rehabilitation strategies at the network level. In essence, pavement management is an optimisation problem that aims to find the best rehabilitation plans that can meet specified performance outcomes and cost constraints. However, the identification and scheduling time of potential rehabilitation plans require the ability to predict the pavement conditions at any given future time. Therefore, a key component for effective pavement management decisions is a reliable performance prediction model such as the discrete-time Markov model presented earlier. The main outcome of performance prediction is the development of a pavement performance curve in terms of a key performance indicator such as PSI and PCI. Several pavement management models developed in the last couple of decades had used some form of performance prediction models. In addition, any pavement management model requires to incorporate an effective rehabilitation module that can effectively specify the potential rehabilitation plan required at a specified service time, and consequently the associated cost.

The simplified approach presented earlier for estimating the overlay thickness schedule from reduced asphaltic thickness represents a potential tool to be incorporated in any pavement management model. Its first main requirement is the availability of performance curves similar to the one shown in Figure 2. Performance curves can be developed using both deterministic and probabilistic prediction models or just from historical distress records. Its second main requirement is an overlay model/procedure that can estimate the overlay thickness schedule as a function of service time and/or other related parameters such as age and pavement condition indicator. These two requirements are sufficient to calibrate and use the presented simplified approach as demonstrated in the sample presentation. Three potential rehabilitation plans can be considered including plain overlay ($H_m = 0$), cold milling and overlay ($H_m > 0$), and reconstruction. Reconstruction can be specified when pavement condition drops below a specified threshold value as suggested by Table 1. Reconstruction typically includes complete removal of existing asphaltic surface, placement of levelling aggregate layer, and placement of new asphaltic surface.

Generally, there are two different optimisation approaches used in pavement management modelling (Jorge and Ferreira 2012, Mathew and Isaac 2014). The first one involves maximising the pavement condition subject to rehabilitation cost constraint(s), while the second one requires minimising the rehabilitation cost subject to pavement condition constraint (s). A simple example of the maximisation approach is presented in Equation (15) considering a group of (N) projects. Equation (15) aims to maximise the overall group strength as a function of the remaining strength factors considering a variable remaining service life of (T−t+1) years. The optimal solution will yield a set of projects (i.e. subset of N) along with their optimal rehabilitation scheduling times while meeting the available budget (B). The rehabilitation cost for the jth project is obtained as a multiplication product of the rehabilitation cost rate, RC(i,j), overlay thickness, h_e(i,j), estimated using the proposed approach, and project surface area, SA(j), in squared metre.

$$\begin{aligned} &\text{Maximize:} Z = \sum_{i=t}^{T}\sum_{j=1}^{N} I(j) \times SA(j) \times [1 - F_s(i, j)], \\ &\quad I(j) = 0 \text{or } 1 \end{aligned} \tag{15}$$

$$\text{Subject to: } \sum_{i=t}^{T}\sum_{j=1}^{N} RC(i, j) \times h_e(i, j) \times SA(j) \le B$$

The RC(i,j) is the rehabilitation cost rate ($\$/m^3$) associated with the jth project and ith service time in years, F_s(i,j) is the remaining strength factor estimated using Equation (5), and I (j) is a binary number (i.e. 0 or 1). The remaining strength factor represents the proportion of strength remaining in the asphaltic surface, therefore maximising the overall group strength requires selecting projects with the lowest remaining strength factors as indicated by Equation (15). Maximising of equation (15) is considered relatively simple task for a moderate group size with (N) projects compared to other complex pavement management models. It also provides a more accurate cost estimate as it yields a distinct overlay thickness for different projects with variable rehabilitation scheduling times. In addition, it accounts for the pavement long-term performance through the estimation of the asphaltic remaining strength factor.

Table 1. Sample overlay and cold milling thicknesses as a function of $\overline{DR}(T-t)$, (H_a=10).

$\overline{DR}(T-t)$	75[a]–70	70–65	65–60	60–55	55–50	50–45	45–40	40–35	35–30	30–25[b]
h_o (cm)	2	2	2.5	3	3.5	4	4.5	5	5.5	6
H_m (cm)	0	1	1.5	2	2.5	3	3.5	4	4.5	5

[a]No rehabilitation is required when DR is higher than 75.
[b]Complete removal of existing asphalt layer is required when DR is below 25.

3. Sample presentation

In this section, two case studies are presented to illustrate the calibration procedure of the proposed approach for estimating the required overlay thickness schedule as a function of reduced asphaltic thickness. The first case study applies the pavement long-term performance defined as a function of the annual average distress rating considering the two sample projects shown in Figure 3. The second case study deploys the surface deflections as estimated using the Caltrans ME approach for a local roadway sample. The proposed calibration procedure requires the availability of reliable overlay model/procedure that can estimate the observed overlay thickness, $h_o(j)$, as a function of pavement age and/or pavement condition indicator. Unfortunately, renowned overlay procedures typically estimate the required overlay thickness based on the current pavement conditions, however, the proposed calibration procedure requires an overlay thickness schedule for variable distress condition over time. Therefore, the authors have selected the 'prescription' procedure with required overlay thicknesses specified according to the predicted distress ratings. The 'prescription' procedure may not be the best to use, but it provides a convenient tool for demonstrating the proposed calibration technique especially in the lack of any other compatible overlay procedures. The 'prescription' procedure is typically used by local governments.

3.1. Case study I: model calibration using long-term performance

The calibration procedure for estimating the overlay thickness schedule from reduced asphaltic thickness is demonstrated using two sample projects. As outlined before, Figure 3 shows the performance curves associated with the two sample pavement projects. The first one represents superior performance as it is associated with increasingly higher deterioration rates [i.e. $P(k)_{1,2}<P(k)_{2,3}<\ldots<P(k)_{9,10}$], whereas the second one indicates inferior performance since it reflects decreasingly lower deterioration rates [i.e. $P(k)_{1,2}>P(k)_{2,3}>\ldots>P(k)_{9,10}$]. Equations (8)-(10) have been used to generate these sample performance curves. The corresponding non-homogenous transition probabilities [i.e. $P(k)_{i,i+1}$] have been estimated using an Empirical-Markovian approach that is a function of the 1st year initial and terminal transition probabilities [i.e. $P(1)_{1,2}$ and $P(k)_{m-1,m}$], traffic load factor reflecting the progressive increase in traffic loading, and strength factor accounting for the progressive decrease in pavement structural capacity (Abaza 2018). However, similar pavement performance curves can also be generated using other deterministic and probabilistic approaches or from historical records of pavement distress. The sample performance curves have been used to compute the two DR averages required to estimate the asphaltic remaining strength factor as defined by Equation (5), namely [$\overline{DR}(T-t)$ & $\overline{DR}(T)$] with the corresponding results provided in Tables 2 and 3.

In addition, the calibration procedure requires the estimation of the observed overlay thickness (h_o) as a function of the pavement performance indicator deployed, consequently the appropriate overlay design procedure can then be selected. In this paper, the average distress rating, $\overline{DR}(T-t)$, over the remaining service life (T−t) is proposed to represent the remaining long-term performance, which is used to estimate the asphaltic remaining strength factor, $F_s(t)$. In this case, the observed overlay thickness is best specified using the 'prescription' procedure which is highly dependent on professional experience and engineering judgement. The prescription method is largely used by local governments. Table 1 provides sample observed overlay thicknesses (h_o) proposed as a function of the average distress rating for an

Table 2. Sample overlay thickness schedule (cm) for superior performance, $\overline{DR}(T)=72.84$.

			$H_a=10$ cm				$H_a=15$ cm			
			(K = 0.40[a], λ = 0.8)				(K = 0.70[b], λ = 0.6)			
t (yrs.)	DR(T − t)	$H_m(t)$[c]	$F_s(t)$	$H''_a(t)$	$h_e(t)$	$h_o(t)$[d]	$F_s(t)$	$H''_a(t)$	$h_e(t)$	$h_o(t)$[e]
5	64.48	1.31	0.952	8.37	1.79	2.31	0.918	13.06	2.13	2.77
6	62.54	1.51	0.941	8.11	2.08	2.51	0.899	12.73	2.49	3.01
7	60.50	1.71	0.928	7.84	2.38	2.71	0.878	12.40	2.86	3.25
8	58.37	1.91	0.915	7.56	2.68	2.92	0.856	12.06	3.23	3.51
9	56.15	2.15	0.901	7.28	3.00	3.14	0.833	11.71	3.62	3.77
10	53.84	2.38	0.886	6.98	3.32	3.38	0.809	11.36	4.00	4.05
11	51.44	2.62	0.870	6.68	3.65	3.62	0.784	11.00	4.40	4.34
12	48.95	2.86	0.853	6.38	3.98	3.86	0.757	10.64	4.79	4.64
13	46.36	3.12	0.835	6.07	4.32	4.12	0.729	10.28	5.19	4.95
14	43.69	3.39	0.815	5.76	4.67	4.39	0.699	9.92	5.59	5.27
15	40.92	3.67	0.794	5.44	5.02	4.67	0.668	9.56	5.98	5.60

[a]Absolute average overlay error = 0.245 cm and minimal SSE = 0.898.
[b]Absolute average overlay error = 0.289 cm and minimal SSE = 1.262.
[c]Estimated using linear equation: $H_m(t)=7.76–0.1\overline{DR}(T-t)$ derived from data given in Table 1.
[d]Estimated using linear equation: $h_o(t)=8.76–0.1\overline{DR}(T-t)$ derived from data given in Table 1.
[e]Estimated using linear equation: $h_o(t)=[8.76–0.1\overline{DR}(T-t)]\times 1.2$.

Table 3. Sample overlay thickness schedule (cm) for inferior performance, $\overline{DR}(T) = 53.73$.

			$H_a = 10$ cm ($K = 0.28$[a], $\lambda = 0.8$)				$H_a = 15$ cm ($K = 0.79$[b], $\lambda = 0.6$)			
t (yrs.)	DR(T − t)	$H_m(t)$[c]	$F_s(t)$	$H''_a(t)$	$h_e(t)$	$h_o(t)$[d]	$F_s(t)$	$H''_a(t)$	$h_e(t)$	$h_o(t)$[e]
5	40.68	3.69	0.925	5.98	4.42	4.69	0.803	10.26	5.21	5.63
6	38.33	3.93	0.910	5.70	4.72	4.93	0.766	9.88	5.63	5.91
7	36.06	4.15	0.894	5.44	5.02	5.15	0.730	9.54	6.01	6.18
8	33.87	4.37	0.879	5.19	5.29	5.37	0.694	9.21	6.36	6.45
9	31.78	4.58	0.863	4.95	5.55	5.58	0.660	8.92	6.69	6.70
10	29.76	4.78	0.848	4.73	5.80	5.78	0.627	8.64	6.99	6.94
11	27.84	4.98	0.832	4.52	6.03	5.98	0.595	8.39	7.27	7.17
12	25.99	5.16	0.816	4.32	6.25	6.16	0.563	8.16	7.52	7.39
13	24.24	5.34	0.800	4.13	6.46	6.34	0.533	7.95	7.75	7.60
14	22.57	5.50	0.784	3.96	6.64	6.50	0.504	7.76	7.96	7.80
15	20.98	5.66	0.768	3.80	6.82	6.66	0.476	7.59	8.15	7.99

[a]Absolute average overlay error = 0.119 cm and minimal SSE = 0.216.
[b]Absolute average overlay error = 0.155 cm and minimal SSE = 0.393.
[c]Estimated using linear equation: $H_m(t) = 7.76–0.1\overline{DR}(T - t)$ derived from data given in Table 1.
[d]Estimated using linear equation: $h_o(t) = 8.76–0.1\overline{DR}(T - t)$ derived from data given in Table 1.
[e]Estimated using linear equation: $h_o(t) = [8.76–0.1\overline{DR}(T - t)] \times 1.2$.

asphaltic surface thickness of 10 cm (H_a). The table also provides the suggested corresponding cold-milling thicknesses (H_m).

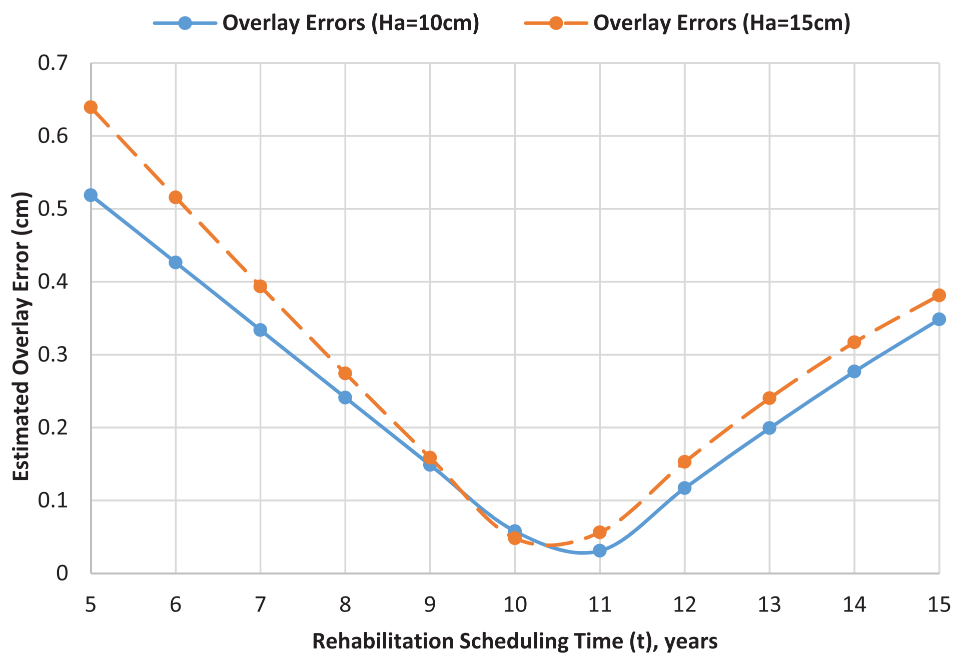
The results of the calibration procedure as obtained from Equations (4)–(7) are provided in Table 2 for the pavement project associated with superior performance. The rehabilitation scheduling time (t) has been varied from 6 to 15 years considering two asphaltic surface thicknesses (10 & 15 cm). The observed overlay thicknesses provided in Table 1 have been increased by 20% in the case of 15 cm asphaltic surface thickness, however, the corresponding cold milling thicknesses assumed to remain unchanged. The vertical distress spread ratio (λ) has been assigned the values of (0.8 & 0.6) for (10 & 15 cm) asphaltic surface thicknesses, respectively. The results provided in Table 2 indicate a good agreement between the estimated and observed overlay thicknesses with (0.70 & 0.40) optimal (K) values and (0.245 & 0.289 cm) absolute average overlay errors considering (10 & 15 cm) asphaltic surface thicknesses, respectively. Table 3 presents similar results for the pavement project with inferior performance. The corresponding optimal (K) values and absolute average overlay errors are (0.28 & 0.79) and (0.119 & 0.155 cm) for (10 & 15 cm) asphaltic surface thicknesses, respectively. The optimal (K) values have been derived using an exhaustive search approach wherein Equations (4)–(7) have been evaluated with the (K) value incrementally increased using one-hundredth point (0.01) in the search for minimal sum of squared errors (SSE). The load factor (F_L) as required by Equation (6) has been assigned (1.10) value assuming (3%) annual traffic growth rate. The load factor can be excluded from the optimisation process but applied to the final results as deemed necessary.

Figure 4 displays the derived sample overlay errors as a function of the rehabilitation scheduling time (t) for both superior and inferior pavement projects. The overall trend is similar to an inverted normal distribution with peak point occurring near the (9–11) years of pavement service time (t) considering the two sample asphaltic surface thicknesses (10 & 15 cm). This represents a positive outcome as pavement rehabilitation most likely will take place around the middle range of the investigated service time, which means lower overlay errors when considering the same pavement project. Figure 4 also indicates that the overlay errors are relatively lower in the case of (10 cm) asphaltic thickness. Figure 5 depicts a perfect-linear direct relationship between the reduced asphaltic thickness H''_a and remaining strength factor, $F_s(t)$, as derived from the sample results provided in Tables 2 and 3. However, the relationship between the estimated overlay thickness (h_e) and remaining strength factor is a perfect-linear inverse one. Figure 6 shows similar trends but for the case of 15 cm asphaltic surface thickness. Figures 5 and 6 indicate a perfect-linear inverse relationship between the reduced asphaltic thickness and estimated overlay thickness as evidenced from the corresponding coefficients of determination (R^2), which are exactly the same when considering the same case. This is expected because any reduction in the reduced asphaltic thickness has to be compensated by an equal increase in the overlay thickness. Also, Figures 5 and 6 show that the estimated overlay thicknesses are evidently higher in the case of inferior performance. Finally, the sample results indicate that the $F_s(t)$ values are generally lower in the case of 15 cm asphaltic thickness compared to the corresponding values in the case of 10 cm surface thickness, and they are generally lower in the case of inferior performance.

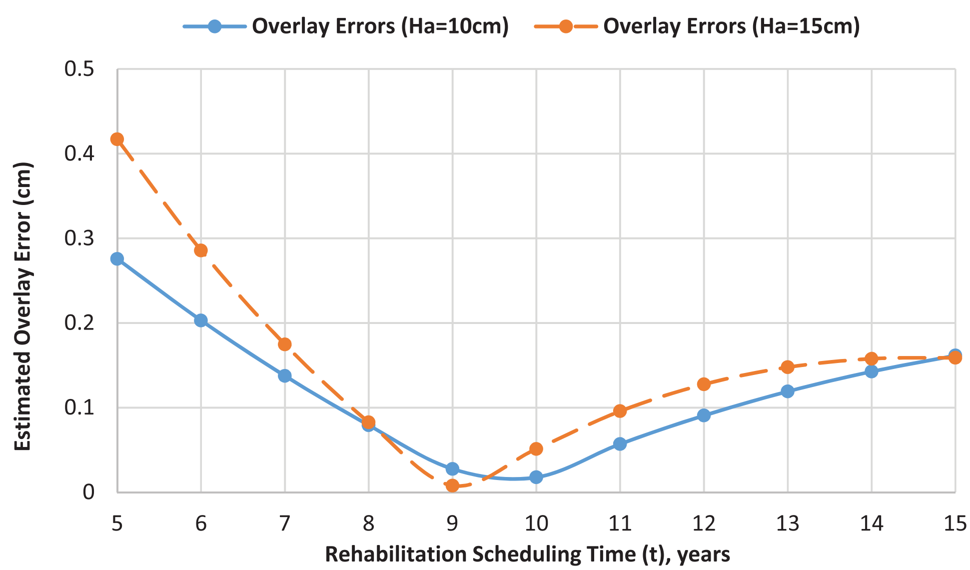
3.2. Case study II: model calibration using surface deflections

The asphaltic remaining strength factor, $F_s(t)$, has been calibrated using pavement surface deflections for a sample of local roads as required by Equation (13). The surface deflections are typically field measured using the FWD method. However, the authors have estimated the surface deflections using the Caltrans mechanistic-empirical (CALME) design approach in a backward solution using the observed overlay thicknesses (h_o) estimated from the 'prescription' procedure outlined earlier. This is because the authors have no access to a Falling-Weight Deflectometer. According to CALME method, the percent reduction in surface deflection, PRD(j), associated with the jth project is computed using Equation (16) as a function of the average surface deflection, $\bar{D}_{80}(j)$, and tolerable deflection at the surface, TDS(j), in inches (Caltrans 2017).

$$PRD(j) = \left(\frac{\bar{D}_{80}(j) - TDS(j)}{\bar{D}_{80}(j)}\right) \times 100\% \tag{16}$$



a) Superior pavement long-term performance.



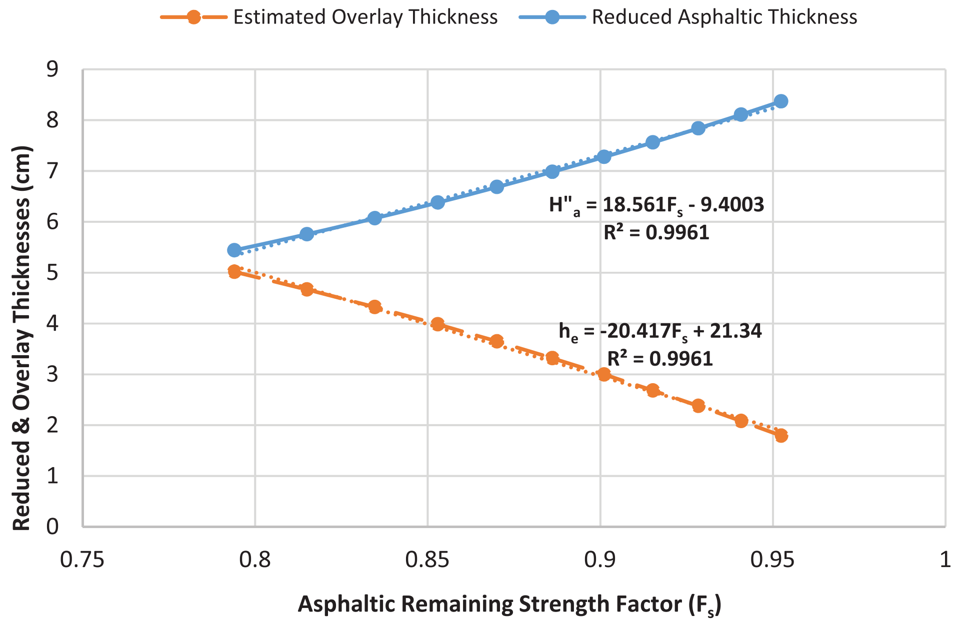
b) Inferior pavement long-term performance.

Figure 4. Sample overlay errors derived from the minimisation of sum of squared errors (SSE).

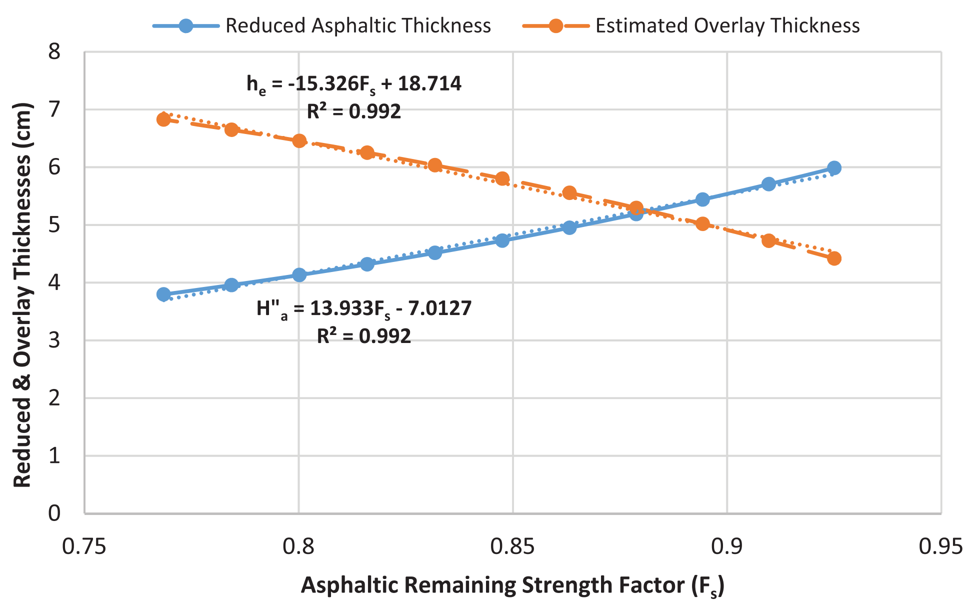
In a backward solution, the percent reduction in surface deflection, PRD(j), has been estimated from consulting a design table that is a function of the overlay gravel equivalent, $GE_o(j)$, computed using Equation (17) based on the observed overlay thickness, $h_o(j)$. Caltrans (2017) recommends a value of 1.9 for the gravel equivalent factor (Gf_o) associated with HMA overlay. A conversion factor is applied to Equation (17) to convert the overlay thickness from (cm) to (ft). The observed overlay thicknesses for the local road sample, $h_o(j)$, as provided in Table 4 have been used to estimate the PRD(j) for the purpose of computing the average surface deflections, $\bar{D}_{80}(j)$, from Equation (16). Therefore, the PRD(j) values provided in Table 5 are mainly obtained as a function of the overlay gravel equivalent, $GE_o(j)$, in feet computed using Equation (17). Table 4 provides other needed design parameters including existing asphaltic surface thickness (H_a), design load applications (W_{80}), and estimated vertical distress spread ratio (λ).

$$GE_o(j) = \frac{Gf_o \times h_o(j)}{30} \tag{17}$$

The tolerable deflection at the surface, TDS(j), for each project is also obtained from consulting another design table as a



a) Superior pavement long-term performance (H_a=10cm)



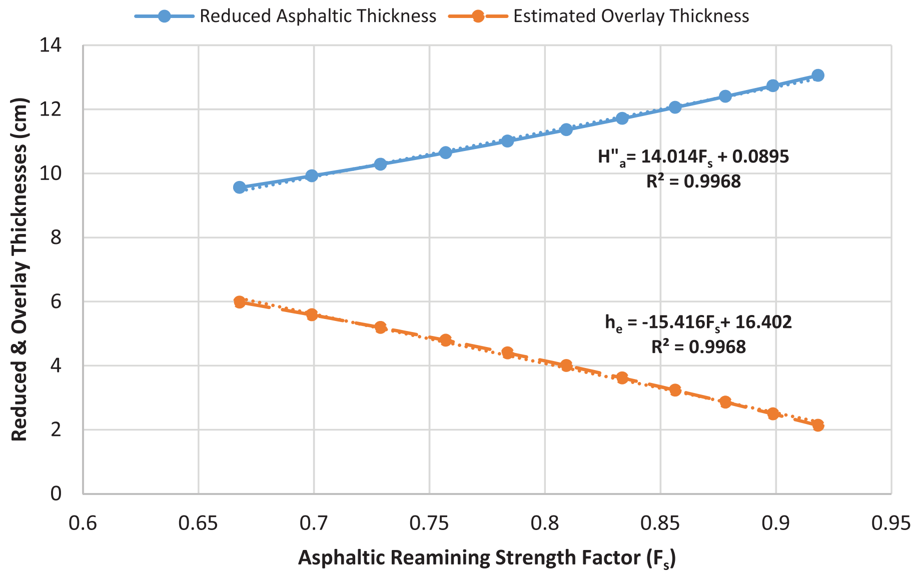
b) Inferior pavement long-term performance (H_a=10cm)

Figure 5. Sample reduced asphaltic and estimated overlay thicknesses as a function of asphaltic remaining strength factor ($H_a = 10$ cm).

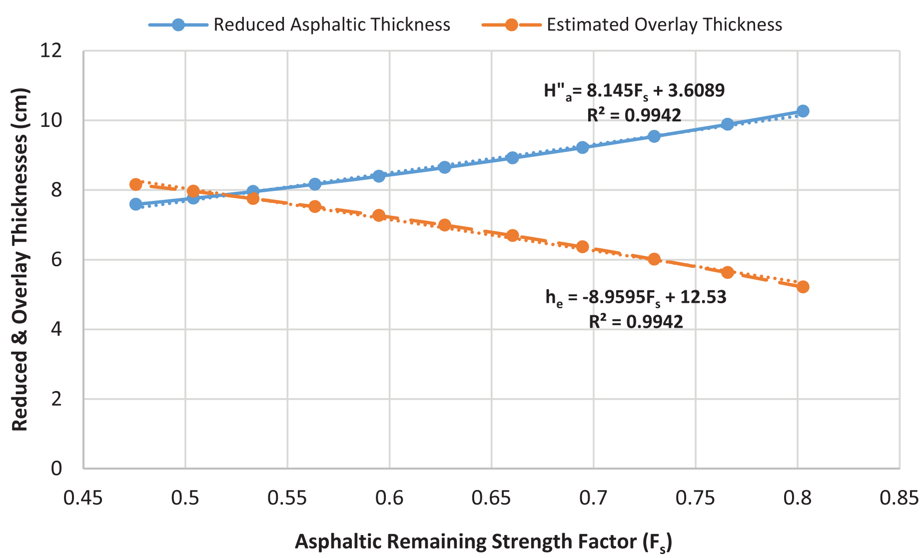
function of the existing asphaltic thickness, $H_a(j)$, in feet, and design traffic index, TI(j), with the corresponding results provided in Table 5. Equation (16) has then been solved for the project average deflection, $\bar{D}_{80}(j)$, with results provided in the same table. The remaining strength factor model as defined in Equation (13) has been calibrated using the minimisation of SSE procedure outlined by Equation (18) considering a roadway sample of size (N). The roadway sample consists of 12 local roads ($N = 12$). The corresponding estimated overlay thicknesses, $h_e(j)$, are computed using Equations (4) & (6) with the load factor (F_L) assigned (1.10) value.

$$\text{Minimize:SSE} = \sum_{j=1}^{N} [h_e(j) - h_o(j)\ ]^2 \tag{18}$$

The minimisation of SSE procedure has resulted in 1.20 optimal (K) value and 0.122 cm absolute average overlay error as provided in Table 5. This case study indicates the effectiveness of using the asphaltic remaining strength factor in estimating the overlay thickness as proposed in this paper.



a) Superior pavement long-term performance (H_a=15cm)



b) Inferior pavement long-term performance (H_a=15cm)

Figure 6. Sample reduced asphaltic and estimated overlay thicknesses as a function of asphaltic remaining strength factor ($H_a = 15$ cm).

However, it is based on surface deflection measurements that are only accessible to state highway agencies.

4. Conclusions and recommendations

A simplified useful approach has been presented to estimate the overlay thickness schedule as a function of the reduced asphaltic thickness considering the two major rehabilitation plans, namely plain overlay, and cold milling and overlay. The main parameter used in the simplified approach is the asphaltic remaining strength factor proposed to be estimated either from pavement performance curves or surface deflections. The first case study has provided reliable results for two sample projects with known pavement performance curves. The proposed calibration procedure based on minimising the sum of squared errors (SSE) has yielded overlay thicknesses with (0.245 & 0.289 cm) absolute average errors for (10 & 15 cm) asphaltic surface thicknesses, respectively, in the case of superior performance. However, the absolute average overlay errors are (0.119 & 0.155 cm) for (10 & 15 cm) asphaltic surface thicknesses, respectively, in the case of inferior performance. The second case study has deployed surface deflections in estimating the remaining strength factor and resulted in (0.122 cm) absolute average overlay error when considering a sample of 12 local roadways. The sample overlay errors associated with both case studies are relatively low, an indication of the effectiveness of the proposed simplified approach.

Table 4. Design and rehabilitation parameters for the local roadway sample.

Road j	$H_a(j)$ (cm)	DR (j)	$H_m(j)$ (cm)	$h_o(j)^a$ (cm)	$GE_o(j)$ (ft)	$W_{80}(j)$ ($\times10^3$)	TI (j)	$\lambda(j)^b$
1	8	73.6	0.0	2.5	0.16	470	8.2	0.88
2	8	67.9	1.0	3.0	0.19	320	7.8	0.88
3	8	59.2	2.0	4.0	0.25	350	7.9	0.88
4	7	61.2	1.5	3.5	0.22	180	7.3	0.92
5	7	53.4	2.5	4.5	0.25	250	7.6	0.92
6	8	62.3	1.5	3.5	0.22	520	8.3	0.88
7	8	54.2	2.5	4.5	0.25	650	8.6	0.88
8	8	48.0	3.0	5.0	0.28	700	8.6	0.88
9	9	41.9	3.5	5.5	0.35	620	8.5	0.84
10	8	46.3	3.0	5.0	0.32	580	8.4	0.88
11	9	43.2	3.5	5.5	0.32	660	8.6	0.84
12	9	41.1	3.5	5.5	0.35	750	8.7	0.84

[a]Estimated based on DR(j) values using the prescription procedure.
[b]Estimated using linear equation: $\lambda = 1.2–0.04H_a$ derived from the two (λ, H_a) points of (1, 5 cm) & (0.6, 15 cm).

Table 5. Sample estimated overlay thicknesses using average surface deflection, $\bar{D}_{80}(j)$.

Road j	PRD (j)	TDS(j) (0.001in)	$\bar{D}_{80}(j)$ (0.001in)	$F_s(t)$ (cm)	$H''_a(t)$	$h_e(j)^a$ (cm)	$h_o(j)$ (cm)	$E(j)^b$
1	24	22	29	0.726	6.03	2.18	2.5	−0.31
2	26	24	32	0.716	5.24	3.04	3.0	0.04
3	30	23	33	0.658	4.22	4.15	4.0	0.15
4	28	28	39	0.681	3.74	3.43	3.5	−0.07
5	30	27	39	0.653	2.94	4.30	4.5	−0.20
6	28	22	31	0.672	4.63	3.71	3.5	0.21
7	30	21	30	0.661	3.91	4.49	4.5	−0.01
8	32	21	31	0.636	3.47	4.96	5.0	−0.04
9	36	20	31	0.601	4.02	5.47	5.5	−0.03
10	34	21	32	0.613	3.38	5.06	5.0	0.06
11	34	20	30	0.625	4.11	5.57	5.5	0.07
12	36	19	30	0.589	3.98	5.73	5.5	0.23

[a]Optimal K =1.20, minimal SSE = 0.507.
[b]$E(j) = h_e(j) - h_o(j)$, absolute average overlay error = 0.122 cm.

The key requirement for estimating the reduced asphaltic thickness and consequently the corresponding overlay thickness is the estimation of the asphaltic remaining strength factor. Therefore, it is recommended that local governments use the outcomes of pavement distress assessment to construct performance curves similar to the ones presented in the first case study. The performance curve can be developed for a particular pavement project or project group with similar pavement structures and traffic loadings. The required performance curves can be predicted using either deterministic or probabilistic approaches mainly replying on historical pavement distress records. Once the performance curves are available, then the proposed approach can easily be used with minimal calculation efforts. The calibration procedure can simply be programmed using the 'Excel' software that is available in Microsoft Word. The proposed approach will yield the overlay thickness schedule as a function of service time to be used in pavement management applications. In addition, state highway agencies can use the proposed approach by mainly relying on surface deflection measurements to calibrate and estimate the asphaltic remaining strength factor as demonstrated by the second case study. Pavement deflection records are readily available to state highway agencies and can be used in the calibration procedure.

Disclosure statement

No potential conflict of interest was reported by the authors.

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