Selection of pavement performance models for use in the Portuguese PMS

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Selection of pavement performance models for use in the Portuguese PMS

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This paper compares different pavement performance models (PPMs) from around the world and recommends one for use in the Portuguese pavement management systems (PMS). The paper is divided into three complementary sections. The first section describes several PPMs used around the world. The models analysed are (or come from) the Highway Development and Management System, the American Association of State Highway and Transportation Officials, the Nevada PMS, the Collop–Cebon whole-life pavement performance, the Swedish PMS and the Spanish PMS. The second section compares the results obtained for a case study with those of the two most promising PPMs and presents the advantages and disadvantages of using these models. The models were tested using the strategies evaluation tool, one of the components of the PMS utilised by the main Portuguese concessionaire. The final section of the paper presents the conclusions of the study on the use of the PPM evaluated and recommends one model to be used for managing the Portuguese highway network.

Keywords: pavement management systems; deterministic pavement performance models; pavement maintenance and rehabilitation; optimisation models; genetic algorithms

1. Introduction

In some forums, the term ‘performance models’ are only related to those models that predict a pavement quality indicator that relates to user perception. This is not the case in this paper where the meaning extends to prediction of pavement condition. With this in mind, pavement performance models (PPMs) are basic components in pavement management systems (PMS). The history of PMS, briefly summarised by Thompson (1994) and Markow (1995), dates back to the late 1960s, and involves what may be considered one of the first applications of computing technologies to civil engineering. A PMS can be defined as a set of tools which helps a road network administration determine the best maintenance and rehabilitation strategies for keeping the pavements in good service condition. A PMS typically includes several modules. For example, Figure 1 shows a ‘generic’ PMS with five modules: a road network database, a quality evaluation system (also referred to as need analysis), a costs model, PPMs and a maintenance strategies optimisation system. The PPM module constitutes one of the basic components of the PMS which is essential for allocating resources, and establishing medium- and long-term maintenance, preservation and rehabilitation plans, with the objective of minimising costs or maximising quality. The best approach to achieve these objectives relies heavily on the prediction of pavement performance and life-cycle cost analysis (LCCA) of candidate maintenance, preservation and rehabilitation strategies.

According to the World Road Association (Ferreira et al. 1999), a PPM is a mathematical representation that can be used to predict the future state of pavements, based on current state, deterioration factors (traffic and climate), and effects resulting from maintenance and rehabilitation actions (or simply M&R actions).

PPMs can be divided into deterministic and probabilistic (Li et al. 1996, Ferreira and Picado-Santos 2004, Stephenson et al. 2004). Deterministic models can be subdivided into purely mechanistic (based on physical models), purely empirical and mechanistic–empirical. Probabilistic models are purely empirical and are defined by transition probability matrices with probabilities of transition between quality states of the pavement with or without application of maintenance and rehabilitation actions (Ferreira et al. 2002a). PPMs can also be classified into network or project-level models (Mbwanza and Turnquist 1996, Ferreira and Picado-Santos 2004). At network-level, performance models are used to predict the future condition of pavements. This information is essential for the definition of medium- and long-term M&R actions to be taken on the pavements of the road network. At project-level, performance models are used to evaluate alternative pavement design strategies, in order to find the most cost-effective solution for specific sections of the road network.

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PPMs can finally be classified as relative or absolute (EC 1997, Ferreira and Picado-Santos 2004). Relative models are those used for predicting the future condition of the pavement based on measured condition data (i.e. bearing capacity, defects, longitudinal roughness, skid resistance, cracking and rut depth). Usually, relative models have only one independent variable, time or traffic. Absolute models are those that include independent variables explaining the pavement deterioration process (i.e. layer thickness, resilient modulus, asphalt characteristics, climate, pavement response and so on).

The development and implementation of relative or absolute models depend, among other things, on the required use of the model and whether or not data for independent variables are available.

Tremendous efforts have been made to improve the methodologies of PPM since the AASHO road test American Association of State Highway and Transportation Officials (AASHTO 1962). PPMs can be developed using the following techniques: regression analysis, Bayesian methodology, homogeneous Markov process, non-homogeneous Markov process and semi-Markov process. Regression analysis allows the development of deterministic PPMs, whereas the homogeneous Markov process, non-homogeneous Markov process and semi-Markov process allow developing probabilistic models. The Bayesian methodology can be used in both deterministic and probabilistic models. Emerging technologies, such as artificial neural networks, fuzzy systems and hybrid soft computing systems, are also gaining ground because of the many uncertainties involved in the prediction of pavement performance (Flintsch and Chen 2004). However, the application of these technologies in general is still in the research and development phase.

2. Pavement performance models analysed

In 1990, the Portuguese Road Administration, called Junta Autónoma de Estradas at that time, initiated the implementation of a PMS considering a probabilistic PPM (Golabi and Pereira 2003). In 2003, the Portuguese Road Administration decided to implement a new PMS considering deterministic PPMs (Picado-Santos and Ferreira 2007, 2008). The main objective of the present study is the evaluation of the most appropriate deterministic PPM to be used in the PMS of the main Portuguese concessionaire (EP, Estradas de Portugal, S.A.), the institution that acted at the time as the Portuguese Road Administration. The methodology used was to analyse a large set of PPMs used in PMS of road administrations from around the world, chose the most promising ones for further evaluation, and to select one for implementation. The main models analysed were the following (Ferreira et al. 2008): the models used in the Highway Development and Management System (HDM-4), the 1993 AASHTO pavement design method, the PMS of the Nevada Department of Transportation, the Collop–Cebon whole-life PPM (WLPPM) used in New Zealand and the Swedish and Spanish Road Administrations PMS.
2.1 Highway Development and Management System models

The Highway Development and Management model (HDM-4) is the successor to the World Bank Highway Design and Maintenance Standards model (HDM-III), which was used by various road agencies all over the world for the last 20 years (Mrawira et al. 1999, Odoki and Akena 2008). The HDM-4 was developed as a support system for decision making for highway administrators and engineers to predict the economic, social and environmental impacts that might occur when making road investment decisions (PIARC 2000, Jain et al. 2005, Parida et al. 2005). HDM-4 considers PPMs for several pavement distresses, including cracking, potholes, rutting and roughness, separating, in general, the initiation phase from the progression phase. For example, it considers a cracking initiation model and a cracking progression model. The pavement condition and change in condition are predicted year by year for each mode of distress in the following sequence (Attoh-Okine and Paris 2005): (a) the surface age for initiation of all cracking and increment in area of all cracking, which are functions of surface type, equivalent single axle loads (ESALs) and pavement strength; (b) initiation and increment in the area of all potholes, which are functions of existing surface distress and the annual number of passing vehicles; (c) increment in rut depth (mean and standard deviation), which is a function of the strength, ESALs, age, cracking, precipitation, rehabilitation status and pavement compaction and (d) increment in roughness, which is a function of strength, ESALs, age, environment, cracking, roughness and changes in rut depth. These PPMs were developed based on the results of large-scale field experiments conducted in different but not all conditions. Consequently, the default equations in HDM-4, if used without calibration, would predict pavement performance that might not accurately match the observed values on road sections. A fundamental assumption made prior to using HDM-4 is that the PPMs should be calibrated to reflect the observed rates of pavement degradation on the roads where the models are applied. Jain et al. (2005) presented a study of calibration of the HDM-4 PPMs for a National Highway Network located in two states of India using data collected for cracking, ravelling, potholing and roughness. The investigation concluded that the PPMs incorporated in the HDM-4 system need to be calibrated for local conditions before being used in any country. An attempt to calibrate the HDM-4 models using data about pavement degra-

dations stored in the Portuguese Road Administration database was made, but it was not possible to calibrate the models due to the limited available data.

2.2 AASHTO pavement performance model

The 1993 AASHTO flexible pavement design method (AASHTO 1993) is the most widely used method in North America and is probably the most widely used method in the world (C-SHRP 2002). This design approach applies several factors such as the change in present serviceability index (PSI) over the design period. For flexible pavements, the method considers the predicted equivalent (80 kN) single axle load applications, materials properties, drainage and environmental conditions, and performance reliability to obtain a measure of the required structural strength through an index known as the structural number (SN). The SN is then converted to pavement layer thicknesses according to layer structural coefficients representing relative strength of the layer materials. The basic design equation of the AASHTO model used for flexible pavements is formulated as follows (AASHTO 1993):

$$\log_{10}(W_{18}) = Z_R S_0 + 9.36 \log_{10}(SN + 1) - 0.2$$

$$+ \frac{2.32 \log_{10}(M_R)}{0.40 + \frac{2.32}{1994} \frac{PSI}{(SN+1)^{1.97}}} + 8.07$$

(1)

and

$$SN = \sum C_n^e C_n^d H_n$$

(2)

where $W_{18}$ is the number of 18-kip (80 kN) ESAL applications estimated for a selected design period and design lane, $Z_R$ is the standard normal deviate, $S_0$ is the combined standard error of the traffic prediction and performance prediction, $\Delta PSI$ is the difference between the initial or PSI0 and the terminal serviceability index (PSI), SN is the structural number indicative of the total required pavement thickness and is given by Equation (2), $M_R$ is the sub-grade resilient modulus (pounds per square inch), $C_n^e$ is the layer (structural) coefficient of layer $n$, $C_n^d$ is the drainage coefficient of layer $n$ and $H_n$ is the thickness of layer $n$ (inch).

Equation (1) can be transformed into Equation (3) to be used directly in the prediction of the PSI value in each year of the planning period.

$$\text{PSI}_f = \text{PSI}_0 - (4.2 - 1.5 \times 10^{-2}) \left(\log_{10}(W_{18}) - Z_R S_0 - 9.36 \log_{10}(SN + 1) + 0.2 - 2.32 \log_{10}(M_R) + 8.07\right) \left(0.4 + \frac{1994}{SN^{1.97}}\right)$$

(3)
This PPM is a good candidate for testing in the Portuguese PMS because it is based on the PSI, a quality index already established for use within the new Portuguese PMS as a measure of pavement quality. This PPM can be represented in Figure 2 using the PSI as a function of the number of 80 kN ESAL applications. For an estimated incremental change in load applications, \((\Delta W_{80})_{t-1}\), corresponds to an incremental change in the PSI \((\Delta PSI)_{t-1}\) and, at the same time, corresponds to an incremental service time interval \((\Delta T_{t-1})\). The PSI in year \(t\) (PSI\(_t\)) is defined as the difference between the serviceability index in year \(t-1\) (PSI\(_{t-1}\)) and the incremental change in the PSI \((\Delta PSI)_{t-1}\). At the same time, PSI is defined as the difference between the initial serviceability index (PSI\(_0\)) and the total incremental change in the PSI \((\Delta PSI)_{t}\). PSI ranges between its initial value of about 4.5 (value for a new pavement) and the AASHTO lowest allowed PSI value depending of the road class (value for a pavement in the end of its service life).

2.3 Nevada pavement performance models

A total of 16 PPMs were developed in the PMS of the Nevada Department of Transportation to cover the most common maintenance and rehabilitation actions used in all Nevada districts. The objectives of these models are to project the performance of pavement sections under the combined influence of traffic and environment. A performance model uses traffic, environment, materials and hot-mix asphalt data in conjunction with actual performance data, measured by the PSI, to predict the long-term performance of pavement sections with application of maintenance and rehabilitation actions. A typical performance model for asphalt concrete overlays used in the Nevada PMS is formulated as follows (Sebaaly et al. 1996, 1999):

\[
PSI = -0.83 + 0.23D_{ov} + 0.19P_{mf} + 0.27 SN \\
+ 0.078T_{min} + 0.0037N_{ftc} - 7.1 \\
\times 10^{-7} ESAL - 0.14Y,
\]

where PSI is the present serviceability index, \(D_{ov}\) is the depth of overlay, \(P_{mf}\) is the percentage mineral filler, SN is the structural number indicative of the total required pavement thickness, \(T_{min}\) is the average minimum annual air temperature (°F), \(N_{ftc}\) is the number of freeze–thaw cycles per year, ESAL is the equivalent single axle load applications and \(Y\) is the year of performance (year of construction is zero).

This PPM is also a good candidate to test in the Portuguese PMS because it also uses PSI as the pavement quality index. In addition, it incorporates structural and environmental factors that cover a wide range of structural packages and climatic conditions, which allow dealing with all the situations encountered in the Portuguese highway network.

2.4 Collop–Cebon whole-life pavement performance model

The Collop–Cebon WLPPM attempts to include the important mechanisms of pavement surface degradation also using a deterministic approach. The WLPPM is capable of making deterministic predictions of fatigue, rutting and surface roughness throughout the life of a pavement. The initial inputs to the WLPPM are (Collop and Cebon 1995a, 1995b, 1996, Pont et al. 1998) (a) the characteristics of the pavement, i.e. layer thicknesses, mix specifications, etc.; (b) the time increment to be used in the simulation; (c) the rate of traffic loading and (d) the climatic conditions under which the pavement is...
monitoring the change in project condition, deterministic PPMs are used based on historical measurements of roughness and rut depth. In the Swedish PMS, different PPMs for four climate zones and eight traffic classes were defined. A time series of roughness and rut depth to identify candidate road projects for each homogeneous road section can be created. The PPMs used in the Swedish PMS for flexible pavements and traffic class 4 (1000 ≤ AADT < 2000) are formulated as follows (Lang and Dahlgren 2001, Lang and Potucek 2001, Ihs and Sjögren 2003, Andersson 2007):

\[
\text{IRI}_t = \text{IRI}_0 + \Delta \text{IRI}_t Y_t, \quad (5)
\]

\[
\text{RD}_t = \text{RD}_0 + \Delta \text{RD}_t Y_t, \quad (6)
\]

\[
\text{IRI}_{t+1} = 1.068788 + 0.266447 \text{IRI}_t - 0.012363 H_t^n, \quad (7)
\]

\[
\Delta \text{IRI}_{t+1} = 0.014645 + 0.114265 \Delta \text{IRI}_t + 0.050106 \text{IRI}_0, \quad (8)
\]

\[
\text{RD}_{t+1} = 4.308123 + 0.070647 \text{RD}_t - 0.05265 H_t^n, \quad (9)
\]

\[
\Delta \text{RD}_{t+1} = 0.987111 - 0.082187 \Delta \text{RD}_t + 0.00904 \text{RD}_0, \quad (10)
\]

where IRI is the International Roughness Index in year \(t\) (mm/km), \(\text{IRI}_{t+1}\) is the IRI in year \(t + 1\) (mm/km), \(\text{IRI}_0\) is the IRI in the year of construction or the last rehabilitation (mm/km), \(\Delta \text{IRI}_t\) is the yearly change in the IRI in year \(t\) (mm/km), \(Y_t\) is the age of the pavement in year \(t\) since the construction or the last rehabilitation (years), \(H_t^n\) is the thickness of a major maintenance intervention (mm), \(\text{RD}_t\) is the rut depth in year \(t\) (mm), \(\text{RD}_{t+1}\) is the rut depth in year \(t + 1\) (mm), \(\text{RD}_0\) is the rut depth in the year of construction or the last rehabilitation (mm) and \(\Delta \text{RD}_t\) is the yearly change in rut depth in year \(t\) (mm). Equations (5) and (6) are used to predict roughness and rut depth when only routine maintenance is applied to the pavement, while Equations (7)–(9) are used when a major maintenance intervention is applied to the pavement.

At the project level, the objective is the definition of maintenance and rehabilitation interventions in existing pavements or the design of new pavements, a linear elastic approach to solve the problem is used with computing tensile stresses in the asphaltic bound layers (horizontal) and on the sub-grade surface (vertical).

The PPMs used by the Swedish PMS at the programming level were considered interesting to test in the Portuguese PMS because they consider roughness and rutting. Nevertheless, since the PSI pavement quality index is used in the Portuguese PMS to establish the maintenance and rehabilitation interventions every year of the planning time-span, besides roughness and rutting modelled by the Swedish approach, PPMs for cracking,
Therefore, the agency had to compute the Pavement Quality Index (PQI) due to the high cost of deflection measures. Distress surveys for evaluating the structural condition of the Spanish National Road Network PMS use surface Pavement Performance Models (PPMs). These PPMs are not interesting to apply in the Portuguese PMS essentially because the falling weight deflectometer is the equipment used to measure the deflection instead of the Curviameter and because it is important to consider other degradations than cracking to characterise the Portuguese pavements, as for example roughness and rutting.

### 2.6 Spanish Pavement Performance Models

The Spanish National Road Network PMS uses surface distress surveys for evaluating the structural condition of pavements due to the high cost of deflection measures (Gutierrez-Bolivar 2001). Therefore, the agency had to study the relation between deflections and fatigue cracking, the principal distress in the Spanish pavements, to justify the use of distress surveys as the method for assigning rehabilitation actions in the PMS. A fatigue cracking index was assigned to each homogeneous road section with 100 m of length. This index represents the length of the lane that presents fatigue cracking. An index value equal to 0 represents a section without fatigue cracks, while 100 denotes that there are cracks along the entire length of the road section. The Curviameter, which takes measurements continuously on a one-wheel track of the lane at a speed of 18 km/h, was used to give a reference measurement of deflection and the curvature radius. Using a regression analysis for fatigue cracking and deflection of each homogeneous road section, the following PPMs represented in Figure 3 were defined (Gutierrez-Bolivar and Achuguetegui 1998, Gutierrez-Bolivar 2001, Gutierrez-Bolivar and Viada 2003):

\[
CI_t^F = D_t^{1.3} \times 10^{-1.10},
\]

where \(CI_t^F\) is the cracking index for flexible pavements with less than 7 cm of asphalt concrete above a granular base (0–100), \(D_t\) is the deflection (10^2 mm), \(CI_t^{F7}\) is the cracking index for flexible pavements with 7–11 cm of asphalt concrete above a granular base (0–100), \(CI_t^{F12}\) is the cracking index for flexible pavements with 12–14 cm of asphalt concrete above a granular base (0–100), \(CI_t^{F15}\) is the cracking index for semi-flexible pavements with 15–20 cm of asphalt concrete above a granular base (0–100), \(CI_t^{SF21}\) is the cracking index for semi-flexible pavements with 20–25 cm of asphalt concrete above a granular base (0–100), \(CI_t^{SR}\) is the cracking index for semi-rigid pavements with asphalt concrete above a cement base (0–100), and \(CI_t^{SR}\) is the cracking index for semi-rigid pavements with asphalt concrete above a cement base (0–100).

\[
CI_t^{F7} = D_t^{1.3} \times 10^{-0.90},
\]

\[
CI_t^{F12} = D_t^{1.3} \times 10^{-0.59},
\]

\[
CI_t^{F15} = D_t^{1.3} \times 10^{-0.53},
\]

\[
CI_t^{SF21} = D_t^{1.3} \times 10^{-0.40},
\]

\[
CI_t^{SR} = D_t^{1.3} \times 10^{-0.49},
\]

\[
CISR = D_t^{1.3} \times 10^{-0.59},
\]

\[
CIF12 = D_t^{1.3} \times 10^{-0.59},
\]

\[
CIF15 = D_t^{1.3} \times 10^{-0.53},
\]

\[
CIFSF21 = D_t^{1.3} \times 10^{-0.40},
\]

\[
CIFSR = D_t^{1.3} \times 10^{-0.49},
\]

\[
F - flexible pavements with less than 7 cm of asphalt concrete above a granular base
F7 - flexible pavements with 7 to 11 cm of asphalt concrete above a granular base
F12 - flexible pavements with 12 to 14 cm of asphalt concrete above a granular base
SF15 - semi-flexible pavements with 15 to 20 cm of asphalt concrete above a granular base
SF21 - semi-flexible pavements with more than 20 cm of asphalt concrete above a granular base
SR - semi-rigid pavements with asphalt concrete above a cement base

Figure 3. Spanish pavement performance curves.

2.6 Spanish pavement performance models

The two most promising PPMs, the AASHTO model and the Nevada model, were tested through the use of the strategies evaluation tool (SET), one of the components of the PMS utilised by the Portuguese concessionaire Estradas de Portugal, S.A. (Picado-Santos et al. 2006a, 2006b, Picado-Santos and Ferreira 2007, 2008). It is based on a deterministic segment-linked optimisation model and solved by a method developed using genetic algorithm principles called GENETIPAV-D (Ferreira 2001, Ferreira 2002a, 2002b, Picado-Santos et al. 2004). The SET is constituted by the components represented in Figure 4: the objective of the analysis, the data and the models concerning the road pavements, the constraints that the system must guarantee and the results. The SET allows the definition of the best M&R actions to be taken in each segment and period, for a given road network and over a given number of time periods.

The AASHTO model was chosen because it presented the following characteristics: (1) it is based on the PSI, a pavement quality index already used in the Portuguese PMS for the evaluation of pavement sections using data from the distress survey; (2) it is used for the design of potholes, ravelling and patching are also needed to
flexible pavements and it is familiar to pavement management and design engineers and (3) the evolution of the pavement quality is influenced by the traffic, the environment, the sub-grade and the pavement structure. The other model chosen for the analysis was the Nevada model, because it also predicts the PSI of pavements but considering other parameters.

Most of the data needed to run the SET with the two PPMs were downloaded from the Portuguese PMS and stored in a specific road database. The remaining data necessary to test the Nevada model were defined in this study and stored in the road database, i.e. the percentage of mineral filler used in the mixture, the average minimum annual air temperature and the number of freeze–thaw cycles per year. The PSI in this study ranged from 0.0 to 4.1, based on the IRI values available in the database, with 4.1 for a new pavement and 0.0 for a pavement in an extremely poor condition. The value 2.0 was considered as the minimum quality level (MQL).

To test the two PPMs using the SET, three different M&R strategies were considered.

(a) Strategy I: corrective-only strategy involving M&R actions triggered when the pavement reaches an MQL,

(b) strategy II: agency costs optimisation approach (corrective–preventive) involving all possible M&R actions and

(c) strategy III: total (agency + user – residual value of pavements) costs optimisation approach (corrective–preventive) involving all possible M&R actions.

The first M&R strategy permits the allocation of corrective M&R actions to pavement sections using the value 2.0 as the MQL in terms of PSI. The objective of strategy II is to minimise the sum of the annual agency costs during the planning horizon. The objective of strategy III is to minimise total costs, computed by adding the annual agency costs and the annual user costs and deducting the residual value of pavements at the end of the planning time-span, while always maintaining the pavements of the entire road network at a PSI value above the MQL of 2.0. The residual value of a pavement expresses the economic value of the pavement structure for the road administration at the end of the planning horizon as a function of the pavement condition at that time.

To analyse the results of the application of the SET using the two PPMs, four pavement sections with different PSI values in the initial year (PSI = 1.79, PSI = 2.15, PSI = 2.75 and PSI = 3.81) were chosen. Table 1 summarises the attributes of these pavement sections. Table 2 presents the M&R actions to be applied to the four pavement sections using the AASHTO model, considering the three maintenance strategies, i.e. (I) M&R operations allocated based on the trigger MQL, (II) M&R operations allocated by the minimisation of agency costs and (III) M&R operations allocated by the minimisation of total costs (agency costs, user costs and terminal value of pavements).

Table 3 presents the M&R actions to be applied in the same four pavement sections but using the Nevada model. Figure 5 represents the predicted evolution of the PSI value over the years for pavement section 05004 of
national road IC8, considering strategy I and the two PPMs: the AASHTO model and the Nevada model. For this pavement section, which is in a reasonable quality condition (PSI value of 2.75), if strategy I is adopted, one can verify that the SET, using the two performance models, recommends the same M&R action 5 (major rehabilitation) but in different years: in the year 2010 considering the AASHTO model; in the year 2013 considering the Nevada model.

Analysing Figure 5, one can verify that, before the intervention, this segment presents a faster degradation considering the AASHTO model than when that for the Nevada model. This is due to the deficient structural capacity of this pavement section (California bearing ratio (CBR) and SN) considering its annual average daily heavy traffic, i.e. the AASHTO model is much more sensitive to the structural variables than the Nevada model (which is probably defined for stronger pavements). Figure 5 also shows that, in the years following the intervention, this pavement section presents a slower degradation according to the AASHTO model than when using the Nevada model. After re-establishing the section’s structural capacity, the AASHTO model deteriorates slower than the Nevada model for the same conditions.

Figure 6 presents the predicted evolution of the PSI value over the years for pavement section 05001, considering strategy I and the two PPMs. For this pavement section, which is in good quality condition

<table>
<thead>
<tr>
<th>Table 1. Main attributes of pavement sections.</th>
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<tbody>
<tr>
<td><strong>Attributes</strong></td>
</tr>
<tr>
<td>Section_ID</td>
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<tr>
<td>Road_class</td>
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<tr>
<td>Pavement_type</td>
</tr>
<tr>
<td>District</td>
</tr>
<tr>
<td>Length (m)</td>
</tr>
<tr>
<td>Width (m)</td>
</tr>
<tr>
<td>Sub-grade_CBR (%)</td>
</tr>
<tr>
<td>Structural_number</td>
</tr>
<tr>
<td>Age_of_pavements (years)</td>
</tr>
<tr>
<td>Annual_average_daily_traffic</td>
</tr>
<tr>
<td>Annual_average_daily_heavy_traffic</td>
</tr>
<tr>
<td>Annual_growth_average_tax</td>
</tr>
<tr>
<td>Truck_factor</td>
</tr>
<tr>
<td>PSI</td>
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<tr>
<td>Percentage_mineral.filler (%)</td>
</tr>
<tr>
<td>Average_minannual_air_temp. (°F)</td>
</tr>
<tr>
<td>Number_of_freeze-thaw_cycles_per_year</td>
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</tbody>
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<thead>
<tr>
<th>Table 2. Programmed M&amp;R actions using the AASHTO model.</th>
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<tbody>
<tr>
<td><strong>Section</strong></td>
</tr>
<tr>
<td>Strategy I. M&amp;R operations allocated based on the trigger MQL</td>
</tr>
<tr>
<td>05012</td>
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<td>05003</td>
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<td>05004</td>
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<td>05001</td>
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<td>Strategy II. M&amp;R operations allocated by the minimisation of agency costs</td>
</tr>
<tr>
<td>05012</td>
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<td>05003</td>
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<td>05004</td>
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<td>05001</td>
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<tr>
<td>Strategy III. M&amp;R operations allocated by the minimisation of total costs</td>
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KEY (M&R actions): 1, do nothing; 2, non-structural surface rehabilitation; 3, light structural rehabilitation; 4, medium structural rehabilitation and 5, major structural rehabilitation.
(PSI value of 3.81), if strategy I is adopted, the SET does not recommend any intervention during the 10 years of the planning horizon using either of the performance models. Figure 6 shows that this segment always presents a slower degradation according to the AASHTO model than according to the Nevada model. Figure 7 presents the predicted PSI value for the same pavement section, but now considering strategy III with the objective of minimisation of total costs, computed by adding the annual agency costs and the annual user costs and deducting the residual value of pavements at the end of the planning horizon. Analysing Tables 2 and 3, one can see that the SET recommends the same M&R action 2 (non-structural surface maintenance) for strategy III using both performance models, but in different years: in the year 2016 considering the AASHTO model and in the year 2015 considering the Nevada model. The segment in Figure 7 shows the AASHTO model always presents a slower degradation than the Nevada model.

4. Conclusions

This paper reviewed several PPMs currently in use in PMS around the world and selected two candidate models for further evaluation. The results obtained by the application of the SET using the two PPMs to a case study permit the following comments:

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<td>Strategy I. M&amp;R operations allocated based on the trigger MQL</td>
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KEY (M&R actions): 1. do nothing; 2. non-structural surface rehabilitation; 3. light structural rehabilitation; 4. medium structural rehabilitation and 5. major structural rehabilitation.
The Nevada model is easier to implement than the AASHTO model and, in general, produces more conservative results. The Nevada model produces acceptable results; however, it is not very sensitive to the traffic or to the structural variable (SN), and some of the other variables considered in the model are not important in respect to Portuguese technology (for instance, the percentage mineral filler);

(2) The AASHTO model also produces acceptable results and predicts, in general, slower degradation than the Nevada model. This model considers more relevant variables than the Nevada model and is more sensitive to the pavement structural capacity (SN).

Based on these results, this study recommends the AASHTO model to be used in an initial phase of implementation of the Portuguese PMS. The same model has been used in an LCCA model in the USA (Chen and Flintsch 2007). The recommended performance model predicts the PSI, a pavement quality index already used in Portugal for the functional evaluation of pavement sections. This index is computed using data from field condition surveys. The model’s functional form can be easily related to pavement condition evolution because the evolution of the pavements quality is influenced by the traffic, environmental indicators, the sub-grade strength and the overall pavement structure. In particular, the model is very sensitive to structural variables (sub-grade CBR and pavement SN). The application to a sample of pavements from the road network of a Portuguese region demonstrated that a slight adaptation of the AASHTO model gives acceptable results in terms of prediction of pavement performance for the Portuguese national network.

When enough pavement condition time series data from field surveys become available, it is recommended that a full verification and validation of both models should be conducted, and the need to develop different PPMs using the regression analysis or other techniques can be re-evaluated.

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