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Performance Prediction Models for Pavement Management with Emphasis on Compatibility of Network and Project Level Solutions

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ABSTRACT

A key component of a Pavement Management System (PMS) that was implemented on several highway networks in Brasil are its performance prediction models. They were developed with the objective of reducing to a minimum a major shortcoming of existing models: the great distance between maintenance solutions established at network level and the ones that are effectively applied after studies at project level. This paper presents the development of these models and their first applications for pavement management of several highway networks. Calibration factors for the models were consistent with varying environmental and construction conditions on these highway networks.

1 - INTRODUCTION

At network level, when a PMS applies performance prediction models they are simplistic, due to the necessity of keeping monitoring field data at low cost per kilometer. Besides, several systems attempt only to detect at one side the pavement segments were minor maintenance measures can be applied, and at the other the ones were a major rehabilitation should be scheduled. These facts lessen the need for truly reliable performance prediction models at network level.

On the other hand, experience with PMS in highway networks in Brasil led to the need for performance models that go further: they should have the ability to anticipate structural needs that will not be grossly over- or underestimated when rehabilitation designs are performed. A complete agreement between network and project level solutions is impossible, since additional factors are usually taken into account at project level, such as localized environmental conditions (drainage, geometry, traffic loads and lateral distribution), pavement history and detailed pavement condition data. However, when managerial characteristics impose the need for a good agreement between these two levels of decision, the performance models to be applied at network level must have adequate accuracy to deal with structural parameters (traffic loads and their interaction with pavement structure). This is particularly important in Brasil due to the extremely varied bearing capacities of the highway pavements, with major extensions of highways operating with incredibly thin pavements for the heavy traffic loads applied.

2 – MODELS DEVELOPMENT

2.1 – General Requirements

Based on this discussion, the following characteristics were judged useful for the performance prediction models:

(1) The model must be able to describe the long-term performance of highway pavements;
(2) The interaction between pavement structure and traffic loads must be accurately dealt with;
(3) Inputs for the model should be the data usually available from network monitoring programs;
(4) The models should be easily calibrated, so that remaining life estimates could be done using pavement past performance;
(5) Pavement condition should be adequately and easily described, as well as its modification through pavement maintenance or rehabilitation.

These requirements were met by employing a model whose functional shape was suggested by FHWA-LTPP data, using as performance indicator the PSI concept and structured around The AASHO Road Test models.

Condition of the pavements were described in this model using the AASHTO Present Serviceability Index (PSI) concept. At a certain moment in the pavement life, the PSI was here determined as the average of the two evaluations that are important to describe the overall condition of a pavement:

(a) An engineering evaluation of surface distress, taking into account the nature, extension and severity of the various defects, as they may affect the pavement structural condition and, therefore, the rate at which further deterioration will take place; and
(b) The user evaluation of riding comfort, at the operational speed associated with the functional class of the highway.

In this way, the PSI was defined by:

$$PSI = \frac{PSI_{SD} + PSI_{RC}}{2} \tag{1}$$

where PSI_{SD} is a value between 0 and 5 associated exclusively with surface deterioration, as evaluated by a pavement engineer, and PSI_{RC} is a value, also between 0 and 5, associated exclusively with the user evaluation of the riding comfort.

In order to minimize undesirable effects of the subjective evaluation of these parameters, they are determined by the formulae:

$$PSI_{SD} = \frac{PSR + PSI_{SDI}}{2} \tag{2}$$

$$PSI_{SDI} = \frac{309{,}22 - 0{,}616 \times SDI}{61{,}844 + SDI} \tag{3}$$

$$SDI = \sum_{i=1}^{n} f_i\, p_i \tag{4}$$

$$PSI_{RC} = 5e^{\left(-QI/71.5\right)} \tag{5}$$

where:

PSR = Present Serviceability Rating (the subjective evaluation of the overall pavement condition in terms of distress);

PSI_{SDI} = has the same objective as the PSR but the evaluator does not chose a value. Rather, it is calculated from a Surface Distress Index (SDI), determined from the nature, extension and severity of the registered surface distresses, as the summation of the products of frequency (f_i) versus weighing factor (p_i) of the *n* existing defects;

QI = the Quarter-Car Index, in counts per km, that measures longitudinal roughness (QI = 13×IRI, where IRI is the International Roughness Index, m/km). Equation (*5*) is from a World Bank experimental study (Paterson, 1987).

Equations (*3*) and (*4*) are from a procedure used in Brasil for highway pavements surface condition evaluation (Pereira, 1979).

2.2 – Asphalt Pavements

As illustrated by Figure 1, analysis of FHWA-LTPP data shows that longitudinal roughness progression can be described by an exponential function of pavement age, at least for the pavement sections where could be observed increases from the initial condition (IRI_0) greater that 1 m/km.

A model for roughness progression can also be written in terms of traffic, expressed as the number of accumulated standard 80 kN axle loads N until pavement age t:

$$\mathrm{IRI(N)} = \mathrm{IRI}_0 e^{\alpha \mathrm{N}} \tag{6}$$

where α is a function of the pavement structural strength and of the environmental conditions (climate, geometry, drainage).

Merging equations (*5*) and (*6*) leads to:

$$\mathrm{PSI(N)} = 5\left(\frac{\mathrm{PSI_0}}{5}\right)^{e^{\alpha N}} \tag{7}$$

where PSI_0 is the initial (new pavement) Serviceability Index. If the structural parameter α is known, the future performance of the pavement can be predicted. In this respect, emphasis must be given to determine how α varies with pavement structural parameters.

The AASHTO Design Guide (1986) model can be applied in this context, since it was derived from a test specially designed to evaluate the traffic-structure interaction. Besides, it is also based on the PSI concept. The AASHTO model is not, however, reliable as a performance prediction model, due to the accelerated nature of the AASHO Road test, where long-term effects could not be evaluated. Therefore, this model will be here applied exclusively with the objective of furnishing a nominal pavement bearing strength, so that to arrive at a first estimate of parameter α, capable of including in the prediction process the key variables related to pavement structure and traffic loads. Second order effects will be taken into account after these key effects are included in the model. In this respect, the AASHTO design model will be here applied in the same context as it is intended for in The AASHTO Guide (1986): to furnish an estimate of the accumulated traffic that is required for a pavement global serviceability loss from a new pavement condition (PSI_0) until pavement "failure" (PSI_f), when a rehabilitation, such as a major asphalt concrete overlay, will be needed. Therefore, the first estimate of parameter α will be given by:

$$\alpha_A = -\frac{1}{W_{18}} \ln\left(\frac{\ln\left(\frac{PSI_0}{5}\right)}{\ln\left(\frac{2.5}{5}\right)}\right) \tag{8}$$

where W_{18} is the 18 kip single axle loads accumulated number of repetitions associated with a serviceability loss from PSI_0 to $PSI_f = 2.5$. According to the 1986 AASHTO Design Guide:

$$W_{18} = 10^{-6}\left[\frac{(SN+1)}{1{,}05}\right]^{9{,}36}\left[\frac{PSI_0 - 2{,}5}{2{,}7}\right]^{\frac{1}{\beta}}\left[\frac{M_R}{3000\,psi}\right]^{2{,}32}$$

$$\beta = 0{,}40 + \frac{1094}{(SN+1)^{5{,}19}} \tag{9}$$

where SN ($= \sum a_i h_i$) is the pavement Structural Number and M_R is the subgrade resilient modulus (psi).

With the most important variables already included in the model by this process, it is time to deal with environmental conditions and material specific parameters. This will be done by defining a calibration factor F_C in:

$$\alpha = \mathrm{Fc} \times \alpha_A \tag{10}$$

A major effort in this research is at establishing the functional dependence of F_C on its controlling variables. The performance prediction model here proposed can be considered reliable only if patterns can be identified of F_C variation with environmental (climatic, geometric, drainage) and construction (materials, construction quality and construction processes) conditions.

In the case of a rehabilitated pavement, this same model can be applied, with the following modifications:

(a) The Structural Number will now be given by: $SN = a_1 H_R$, where $a_1 = 0.44$ is the asphalt concrete overlay structural coefficient and H_R is its thickness (in inches);
(b) The roadbed resilient modulus will now be given by:

$$M_R = 2\left(1-\nu^2\right)\frac{pa}{D_C} \tag{11}$$

where M_R is calculated as the semi-infinite elastic space equivalent modulus of the pavement structure (before rehabilitation), and D_C is the pavement deflection, measured under a circular applied load of pressure *p* and radius *a*. The Poisson's ratio ν of this equivalent system can be fixed at an average typical value for pavement materials (such as 0.33).

The idea behind considerations (a) and (b) is that deterioration of a rehabilitated pavement is essentially the generation of distresses inside the applied asphalt concrete overlay, with the old pavement behaving only as an elastic platform which gives a support described by its equivalent elastic modulus M_R, calculated by equation (*11*). Such consideration will lead to an underestimate of pavement performance only if the old pavement has a significant remaining life, since in this case it cannot be degenerated simply as an "elastic platform". Practical applications of this model in such cases were made by adding an effective overlay thickness to the actual applied overlay thickness H_R.

It is unlikely that the value of F_C for a new pavement can also be applicable to a rehabilitated one, since an existing pavement has already suffered the conditioning effects due to traffic loads, as well as long-term climatic and aging effects. It can be anticipated that F_C for a new pavement must be greater than the value for an overlaid one, since the conditioning effects that the traffic loads apply to all pavement layers lead to a pavement structure more stable than a new one, which has been subjected only to the stress history applied by the compaction equipments. By the other hand, a severely cracked old pavement can lead to the reflection cracking process controlling the rehabilitated pavement performance, which will increase F_C values.

3 – CALIBRATION OF MODELS

HDM-III empirical models will be applied in order to get a first estimate of the calibration factor F_C. After that, the data bases of the highway networks were the PMS was implemented will be employed to analyze possible trends of F_C with structural and environmental conditions.

3.1 – Use of Empirical Models

The HDM-III models (Paterson, 1987) have several shortcomings as true performance prediction models, specially when considering its narrow experimental base and lack of mechanistic considerations, both leading to loss of predicting power. The effects of a change in thickness of the asphalt concrete surfacing layer are not properly dealt with, since its increase only slightly increases pavement service life. Therefore, these models were here applied only for a single reference condition, chosen to represent the average experimental condition of the data base. It is believed that, for such condition, the models predictions are not biased and will reflect the life expectancy for those conditions. These are:

Surfacing layer: asphalt concrete (8 cm)
Granular Base layer with CBR = 80 (15 cm)
Granular sub-base layer with CBR = 30 (27 cm)
Subgrade soil with CBR = 7
Traffic: 3.0×10^5 repetitions per year of the 80 kN single axle load

Such pavement structure has SN = 3.38 and deflection $D_C = 65 \times 10^{-2}$ mm. Assuming an initial condition given by $PSI_0 = 4.5$, Figure 2 shows the performance predicted with HDM-III models. It is also showed that the model here developed agrees with HDM-III predictions for $F_C = 2.5$, at least until a pavement age equal to 15 years. After this period, HDM-III predictions are not reliable, since they indicate an extremely low deterioration rate.

In the case of semi-rigid pavements (the ones with a cemented base layer), the average reference conditions for the HDM-III models were:

Surfacing layer: double surface treatment (2.5 cm)

Cemented Base layer: Soil-cement with M_R = 70000 kgf/cm^2 (17 cm)
Subgrade soil with CBR = 15
Traffic: 3.0×10^5 repetitions per year of the 80 kN single axle load
Pavement deflection: $D_0 = 25 \times 10^{-2}$ mm
Pavement Structural Number: SN = 2.02

Figure 3 shows the comparison between HDM-III models prediction and the performance prediction model here developed, with $F_C = 0.54$ as the best calibration factor.

Applying this same process to a rehabilitated pavement, the average reference conditions are:

Existing pavement deflection: $D_C = 65 \times 10^{-2}$ mm
Asphalt concrete overlay thickness: H_R = 8 cm
Pavement deflection after overlay construction: 41×10^{-2} mm
Subgrade soil with CBR = 7
Traffic: 3.85×10^5 repetitions per year of the 80 kN single axle load
Longitudinal roughness: QI_{old} = 45 counts/km
Area cracked of the old pavement, with severe cracks: 50%

In this case, a calibration factor $F_C = 0.104$ (Figure 4) is required for the agreement between both models. Such calibration factor must be associated with the reflection cracking process controlling rehabilitated pavement performance, due to the severe cracking of the old pavement and the medium to heavy traffic condition. The initial rehabilitated pavement condition ($PSI_0 = 4.3$) was predicted applying the experimental model:

$$QI_0 = 19 + \frac{QI_{old} - 19}{1 + 0.602 H_R} \tag{12}$$

where QI_0 is the initial rehabilitated pavement roughness, with the overlay thickness (H_R) in cm.

3.2 – Pavement Management Systems Data Bases

Table 1 shows the average calibration factors (Fc) for the brazilian highway networks, whose PMS's employ the performance prediction models here presented. The average values and associated standard deviations were calculated for the categories of pavement structures indicated on that table. Each case analyzed comprises an extension around 1 km on a single lane of highway. Total network involved around 5.000 km of highways. In the case of overlaid pavements, conventional asphalt concrete had to be analyzed separately from thinner overlays that constitute micro-asphalt concrete systems, with thicknesses ranging from 5 mm to 12 mm in the latter case.

The average Fc for double surface treatment over a soil-cement base (2.118) was greater than the Fc = 0.54 indicated by HDM-III model. This implies on lower deterioration rates for the HDM-III data base. In fact, the value Fc = 2.118 came from pavements situated on colder regions (sub-tropical), whereas the HDM-III data base had only pavements situated on a tropical climate. It is a well known fact that greater temperature differences between day and night lead to severe transverse thermal cracking of cemented bases. These transverse cracks are the origin of fatigue cracks produced later by the traffic loads.

In the case of asphalt concrete hot mixes over a granular base layer, the HDM-III value (Fc = 2.5) is nearly reproduced by the highway pavements here analyzed, irrespective of climatic conditions. Flexible pavements are not so sensitive to colder climatic conditions effects as are semi-rigid pavements.

In the case of overlaid pavements with asphalt concrete, lower values for Fc than the one from HDM-III were found. It is believed that the major reason for such difference relies on the cracking of the pavement before rehabilitation. The HDM-III models analysis was associated with a severely cracked pavement, whereas the networks here analyzed were subjected to pavement repairs prior to overlay, involving milling and filling of severely deteriorated areas. In the few instances where this was not the case, Fc values approached the HDM-III value.

4 – CONCLUSIONS

The high scatter of the calculated calibration factors for the highways here analyzed is in part due to: different environmental conditions, differences in materials and construction practices, and errors in the documented pavement histories. In spite of this, it can be stated that the models her developed are useful for remaining life predictions and future maintenance needs evaluation, since the model takes into account the major variables involved in pavement performance. Also, the average calibration factors for a highway or region are applied in the pavement management systems only when there is uncertainties about pavement history. Otherwise, pavement past history is employed to calculate the Fc for each pavement section, and its performance is predicted year by year using the differential form of equation (*7*):

$$\delta PSI = \alpha \times \delta N \times PSI \times \ln\left(\frac{PSI}{5}\right) \tag{13}$$

where δN is the traffic applied during the next year, leading to a serviceability decrease from PSI equal to δPSI. Besides, analysis of the calculated Fc values allows the discovery of errors in the reported pavement maintenance history.

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TABLE 1 Synthesis of Calibration Factors for the Brazilian Highways Analyzed

Surfacing Layer	Base	Average Fc	Standard Deviation	Number of Cases	Fc from HDM-III
Double Surface Treatment	Gravel/Soil	0.109	0.139	1060	
Double Surface Treatment	Crushed Stone	0.559	1.461	425	
Double Surface Treatment	Soil Cement	2.118	1.274	168	0.54
Asphalt Concrete - Cold Mix	Gravel/Soil	0.129	0.039	16	
Asphalt Concrete - Cold Mix	Crushed Stone	0.472	0.725	92	
Asphalt Concrete - Hot Mix	Gravel/Soil	1.847	4.574	450	2.50
Asphalt Concrete - Hot Mix	Crushed Stone	2.972	18.364	3459	2.50
Asphalt Concrete - Hot Mix	Soil Cement	41.910	77.621	501	
Asphalt Concrete - Hot Mix	Cemented Crushed Stone	88.878	186.356	511	
Asphalt Concrete - Hot Mix	Bituminous Macadam	15.625	32.852	162	
Asphalt Concrete - Hot Mix	Existing Pavement	3.931×10^{-2}	0.2137	3359	0.104
Micro-Asphalt Concrete	Existing Pavement	4.223×10^{-3}	1.165×10^{-2}	1354	
Double Surface Treatment	Existing Pavement	1.665×10^{-2}	2.036×10^{-2}	196	

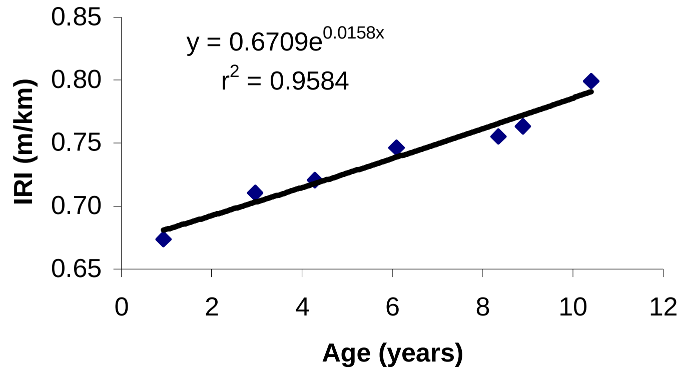

Figure 1 Section SHRP_ID = 1, STATE_CODE = 48 (DataPave 3.0).

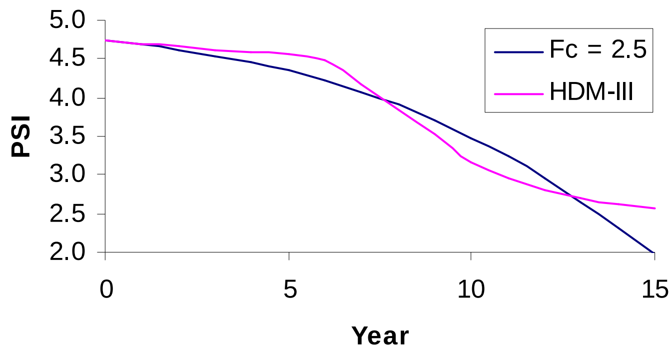

Figure 2 Analysis of HDM-III models for flexible pavements with F_C = 2.5.

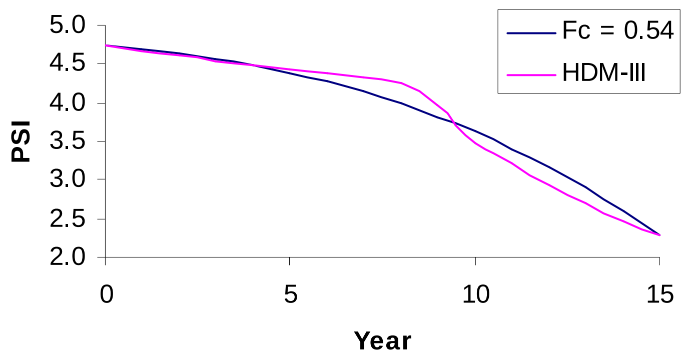

Figure 3 Analysis of HDM-III models for semi-rigid pavements with F_C = 0.54.

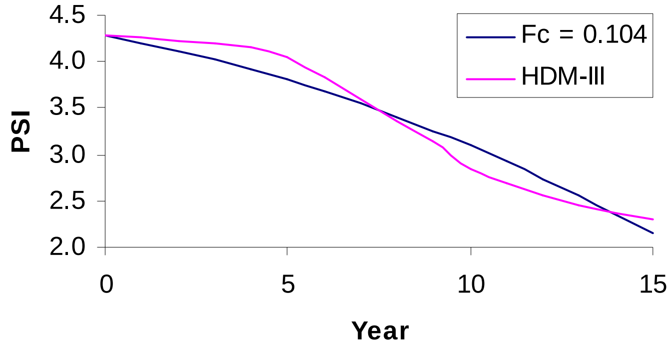

Figure 4 Analysis of HDM-III models for overlaid pavements with $F_C = 0.104$.