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Maintenance and Rehabilitation Programming of the Portuguese Road Network: Development of a Cracking Prediction Model

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Abstract: This paper describes the development of a cracking prediction model for Portuguese conditions which is expected to integrate the Pavement Management System (PMS) of Estradas de Portugal. The World Bank's highway development and management (versions III and 4) and PARIS models are used as reference for the development of a deterministic (mechanistic-empirical) model, using pavement condition data from sections of the main road network. A two-phase distress evolution model is proposed where the initiation of cracking (1st phase) is ruled by a different equation than the progression of cracking (2nd phase). Cracking initiation is predicted on a traffic basis, from the annual traffic load and the structural capacity of the pavement. An absolute model is presented and recommended for the maintenance and rehabilitation (M&R) programming in the long-term and for the analysis of non-cracked segments. Absolute and relative type models were obtained for cracking progression. The relative model shows better agreement to data and is proposed for short- to medium-term analysis on segments with cracking history, while the absolute model is proposed for the M&R programming in the long-term and the analysis of non-cracked segments. Finally, the recommended model is evaluated based on the application to a set of pavement structures defined in the Portuguese pavement design guide.

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Key words*: Cracking; Flexible pavements; Pavement condition assessment; Pavement deterioration models; Pavement management systems.*

Introduction

A Pavement Management System (PMS) can be defined as an appliance that helps the road network administration in planning and decision making. It is used to optimize the maintenance and rehabilitation (M&R) action plan for keeping pavements in good service condition within constrains: budget, time, resources, etc. Less and softer M&R actions during the pavement service life bring significant savings to the road administration (costs) and to the society (raw materials and energy consumption, time delay due to road works, etc.). One of the key modules of a PMS is the Pavement Performance Model (PPM). A PPM is, according to the World Road Association, a mathematical representation that can be used to predict the future state of pavements, based on current state, deterioration factors and effects resulting from maintenance and rehabilitation actions [1].

PPMs can be classified according to: (i) type of formulation; (ii) conceptual format; (iii) application level and (iv) independent variables [2]. Regarding the type of formulation, the models can be classified in deterministic models or probabilistic models. In deterministic models the pavement condition is given by a single value function of the independent variables in the model. Differently, in probabilistic models the result is a vector of the same nature that not only indicates the likelihood of the pavement being in a particular state, but also the probability of transition to another state of deterioration over time.

The conceptual format of the models can be mechanistic, empirical or empirical-mechanistic. Mechanistic models are derived from pavement mechanics, empirical models are supported on experimental data and the latter model type has characteristics of both types.

PPMs may be used at the network-level to define the maintenance and rehabilitation action plan of the different road sections, considering optimisation of costs and condition, or at the project level to evaluate and compare different pavement solutions for a specific road section project. Not all models are suitable for application at both levels.

Finally, PPMs are classified into absolute models and relative models [3]. In the absolute models are used several independent variables that are related directly with the deterioration process. Differently, the relative models predict the future condition for the pavement from past condition of that segment. Often, it is determined a model for each pavement condition parameter (cracking, IRI, etc.) and uses only one independent variable, time or traffic [2, 3].

During the last four decades many researchers have proposed different deterministic and probabilistic models for predicting pavement condition evolution. Most models are deterministic due to the inherent ability to describe pavement performance quantitatively and to relate present and future conditions to the materials, the traffic and the environmental characteristics using a mechanistic-empirical approach [4]. Probabilistic models generate a statistical distribution for the pavement condition variable, which are often based on the Markov chains, the survival curves or the Bayesian regressions [5].

Currently, in the PMS of Estradas de Portugal S.A. (Portuguese Road Administration) is used the American Association of State

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Highways and Transportation Officials (AASHTO) pavement performance model. This PPM computes the present serviceability index (PSI), a global pavement condition index. The evolution of PSI with time is related with the traffic, the material properties and the drainage and environmental conditions [6]. When the warning level is attained for the global index, the extent and severity of the different distresses restricts the implementation of more cost-effective techniques that could be implemented if the distresses evolution was predicted in separate [7, 8].

At the network-level, PPMs support the development of short and long-term budget requirements and to produce a prioritized list of road sections to intervene based on a limited budget. In a good PMS, maintenance and rehabilitation actions are first defined on the long-term plan and, then, moved to the medium- (up to 5 years) and short-term (up to 2 years) plans with time.

In 2007 and 2009 the Portuguese Government published legislation [9, 10] establishing EP – Estradas de Portugal, S.A., as the global road network concessionaire and the basis of the concession contract. Within this contract it was established that concessionaires have to submit a Quality Control Plan (QCP) and a Maintenance and Operation Manual (MOM) on a regular basis to the supervisor institution, the Portuguese Road Infrastructures Institute (InIR). The QCP defines the limits of pavement condition parameters (rutting, cracking, roughness, etc.) that are allowed at any time of the concession period. Both documents (QCP and MOM) require knowledge of the pavement condition in advance, not only of the general pavement service condition, but also the quantification of the extent and the magnitude of pavement distresses and the actions to be implemented in each situation. When a concessionaire does not fulfil the QCP, InIR can apply a monetary penalty that depend on the severity of the violation and varies in total between €5,000 and €100,000, or in daily values between €500 and €5,000.

Among the different distress types, cracking is one of the most important and, consequently, is considered in all mechanistic-empirical design guides. Cracking is easily seen by road users and administrators. Thus, it is a good indicator of road general quality and health. Experience tells us that when cracking spreads and no action is taken (e.g. crack sealing or new layers added), road pavement quality quickly decreases. Surface cracking allows water to go into the pavement, accelerating bitumen hardening and reducing the granular layers' bearing capacity. Often, it is the trigger to implement maintenance or rehabilitation actions depending on the affected area, severity and cause.

This paper presents a study performed to develop a cracking prediction model for the Portuguese road network from internationally used models, with a robust background. The proposed model is based on the World Bank's Highway Design and Maintenance Standards Models (HDM) and the models proposed in the PARIS study. HDM models are among the most commonly used models in PMSs all over the world while the PARIS models were developed with data from many European countries. The model takes into account the particularities of pavement condition data (collection methodologies) of EP-PMS. The variables used in the model are recognized to affect cracking and the constants were determined to meet Portuguese conditions.

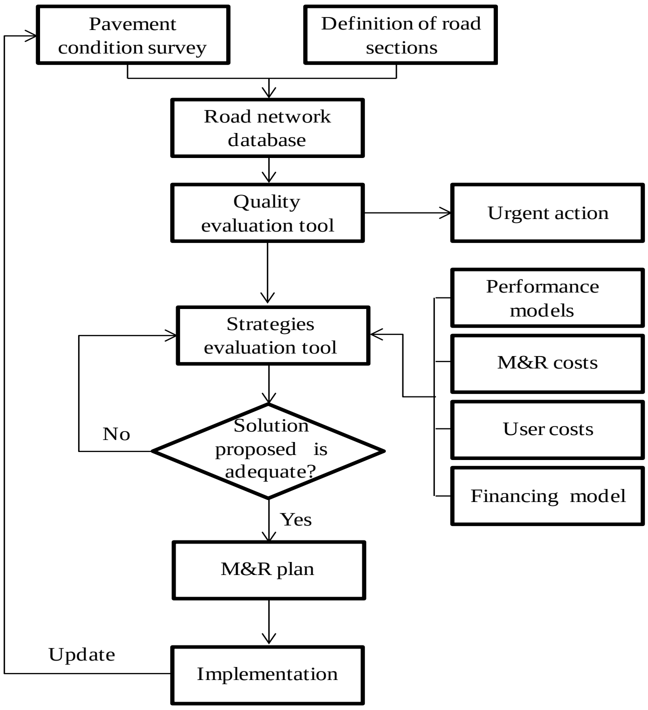

Fig. 1. Framework of the Pavement Management System of EP.

Portuguese Pavement Management System (EP-PMS)

Estradas de Portugal, S.A. has a total road concession of 14,000 km and is currently using a pavement management system (EP-PMS) which was mainly developed in the 2003-2007 period, and that supersedes a previous system (PPMS) developed during the 1990's. Fig. 1 presents the framework of EP-PMS, which includes the basic modules of a common PMS: the road network database; the quality evaluation tool; the pavement performance model; the costs and financing models; the maintenance strategies evaluation tool. This system is planned to integrate the future road asset management system, which includes other infrastructures related with drainage, safety and slopes risk.

The road network database of EP-PMS, the core of a PMS, holds the information about the asset divided in the following three groups: pavement history; traffic; surface condition. The pavement history component describes the pavement structure (type, layers, materials, subgrade, structural number), the pavement condition in the past, including information collected for initial and final handing over, the M&R actions carried out and the cross-section geometry (lane and shoulder widths). The traffic information file includes the annual average daily traffic, total and trucks, determined from last counting and the expected growth rate. The surface condition information file describes the pavement state including the extent and severity of different distress types (see Table 1) obtained in pavement evaluation campaigns at regular intervals.

Up to now, there have been four campaigns for the road network condition assessment, in the years of 2003, 2007, 2009-10 and 2011-2 [11]. A video and laser equipped vehicle was recently acquired for the automatic data collection of pavement condition and it was used in the last campaign. The data available in the previous PMS (PPMS) was not integrated in the database of

Table 1. Pavement Condition Characterization in EP-PMS.

Distress	Severity	Description	Extent
Cracking	Level 1	Single Crack (Visible)	0.5m x Length
	Level 2	Open and/or with Branches Crack (Longitudinal or Transversal)	2.0m x Length
	Level 3	Alligator Cracking	Lane Width x Length
Delamination, Ravelling, Bleeding, Polished Aggregate, Localized Deformations	Level 1	Width < 0.3 m	0.5m x Length
	Level 2	0.3 m < Width < 1 m	2.0m x Length
	Level 3	Width > 1 m	Lane Width x Length
Potholes	Level 1	Maximum Depth < 20 mm	0.5m x Length
	Level 2	20 mm < Maximum Depth < 40 mm	2.0m x Length
	Level 3	Maximum Depth > 40 mm or Several Potholes in the Same Cross Profile	Lane Width x Length
Patching	Level 1	Without Distresses	−
	Level 2	Low Quality Patching or Poor Joint Construction	½ Lane Width x Length
	Level 3	Poor Patching	Lane Width x Length
Rutting	Level 1	Maximum Rut Depth < 10mm	5 mm
	Level 2	10mm < Maximum Rut Depth < 30 mm	20 mm
	Level 3	Maximum Rut Depth > 30mm	30 mm
Longitudinal Unevenness	−	IRI Value	IRI (mm/km)
Skid Resistance	−	SCRIM@60km/h & Mean Texture Depth	Friction Coefficient /IFI/MTD

EP-PMS. The differences in the methodologies used to assess road surface condition (e.g. roughness measured by the inertial-based device APL) make the information of very limited practical use.

The quality evaluation system module determines the indexes used to assess the pavement condition in each section after the road assessment campaign. Currently, a global index similar to PSI is used as quality indicator, which is a modified version of the one developed from the AASHO road test and adopted by the Nevada PMS [11]. The present pavement quality index (IQ) is obtained as follows:

$$IQ = 5 \times e^{-0.0002030 \times IRI} - 0.002139 \times R^2 - 0.03 \times (C_{AL})^{0.5} \quad (1)$$

where IRI is the International Roughness Index (mm/km); R is the rut depth (mm); C_{AL} is the total area with alligator cracking (m^2/100 m^2).

The strategies evaluation tool component allows the definition of the M&R actions to be carried out in each segment and period, using an optimization model based on genetic algorithm principles. The objective-function (minimization) determines the total cost of the administration and users with discount of the pavement terminal value. The model constrains are the annual budget limits and the pavement condition minimum level (distresses individually and IQ). The optimisation calculus uses the costs information and the prediction of the pavements future condition, as a function of the M&R actions taken and the year of intervention.

At the moment, the PPM uses the AASHTO model for PSI (IQ) prediction evolution [12]:

$$IQ_t = IQ_0 - 2.7 \times 10^{\left[log_{10}(N80) - Z_R S_0 - 9.36 log_{10}(SN+1) + 0.2 - 2.32 log10(M_R) + 8.07)(0.4 + 1094 / (SN+1)^{5.19} \right]} \quad (2)$$

where IQ_t is the present quality index at year t; IQ_0 is the initial IQ, after construction; $N80$ is the 80 kN Equivalent Single Axle Load (ESAL) applications for the period; Z_R is the standard normal deviate; S_0 is the combined standard error of the traffic prediction and performance prediction; SN is the AASHTO structural number; M_R is the subgrade resilient modulus (psi). This model was adopted since there are no previously developed models for the Portuguese road network conditions. The main advantage of this option is the fact that the same index is used for the pavement condition assessment [1].

Review of Cracking Prediction Models

Given the relation between the pavement's structural condition, current and future, and the cracking on the pavement surface, for a long time road engineers have paid attention to this distress and collect cracking data in some form [13-15]. However, the mechanisms that cause surface cracking are extremely complex and this presents difficulties with the identification of what should be measured to define cracking in a road section. Paterson [13] suggested the following five attributes:

- extent: area of the pavement with cracks (% or m^2);
- severity: crack width (mm or class);
- intensity: length of cracks per unit area (m/m^2) or crack spacing;
- pattern: crack type related to orientation and interconnection;
- location: part of the pavement affected by cracking.

Most procedures define recording the pattern, the severity and the extent. EP procedures define cracking characterization (see Table 1) with three classes, based on type and severity, and the affected area [16].

Cracking may initiate at the bottom of the bounded layers and propagate upwards, as with fatigue- or reflexion-related cracking, or on the contrary, it may initiate at the surface and propagate downwards, as in structural- or thermal-related cracking. Hence, the process is commonly modelled in two distinct time phases, the time to cracking initiation and the cracking progression, as illustrated in Fig. 2 [13, 17].

Some models such as HDM [13], PARIS [18], Austroads [19], AASHTO [15] follow the two-phases principle. In opposition, other models such as PAVENET-R [20] and KLW [21] simplify the process and consider a single phase.

World Bank Models

The World Bank's Highway Design and Development Standards Model (HDM), version III, was released in 1987 and was the first to have pavement deterioration prediction models as a function of the time, the traffic, the materials, the climate and the maintenance activities [22]. The cracking prediction model was developed with data collected from 1977 to 1982 during the Brazil-UNDP study, and then validated against data from field studies in USA, England, Kenya and Tunisia. During data collection, cracking was classified by the type, the severity class (1 to 4) and the extent (area). However, for the development of the model, the cracking data was separated into two groups, narrow cracks (class 1 and 2) and wide cracks (class 3 and 4), which were used to determine a cracking index representing the pavement's cracking condition. Paterson estimated the area of index cracking (*ACX*) (%) with:

$$ACX = 0.62 \times ACRA + 0.39 \times ACRW \tag{3}$$

where *ACRA* is the area of all cracking (%); *ACRW* is the area of wide cracking (class 3 and 4) (%).

The cracking model comprises two distinct phases, the initiation and the progression. The cracking initiation refers to the time from paving till cracks become visible at the surface, with a minimum cracked area (*Ct*) of 0.5%, as follows:

$$T_{ci} = K_{ci} \times \left[a_0 \times e^{(a_1 \times SNC + a_2 \times N80Y/SNC^2)} + CRT\right] \tag{4}$$

where T_{ci} is the time to structural cracking initiation (years); K_{ci} is the cracking initiation factor (local factor), default 1.0; *SNC* is the modified structural number for the pavement; *N*80*Y* is the indicative annual 80 kN ESAL (million ESAL/lane); *CRT* is the cracking retardation factor (local factor), default 0; a_0, a_1 are model constants.

The modified structural number differs from the AASHTO structural number (SN) by the inclusion of the subgrade contribution, which in the former formulation was considered in the pavement design procedure through the resilient modulus [22, 23]. It is determined as follows:

$$SNC = 0.0396 \sum_{n=1}^{N} [(H_n/25.4) \times C_n^e \times C_n^d] + SNSG \tag{5}$$

$$SNSG = 3.51 \times log(CBR) - 0.85 \times [log(CBR)]^2 - 1.43 \quad if \quad CBR \geq 3\% \tag{6}$$

where H_n is the thickness of layer *n* (mm); C_n^e is the structural coefficient of layer *n*; C_n^d is the drainage coefficient of layer *n*; *SNSG* is the subgrade structural contribution; *CBR* is the California bearing ratio test result (%).

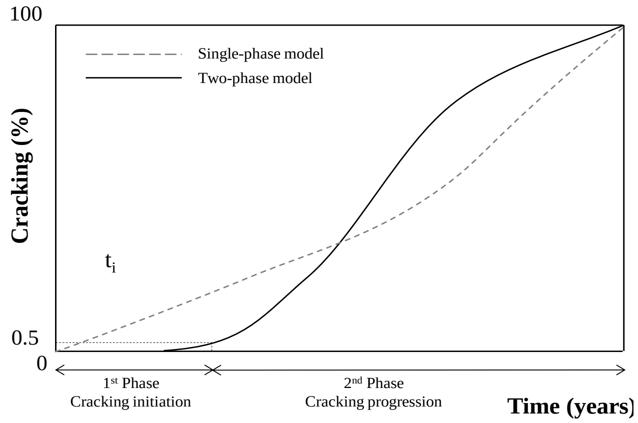

Fig. 2. Cracking Evolution Modelling.

The cracking progression model is based on a sigmoidal (S-shaped) function:

$$C_t = (1-z) \times 50 + z \times \left[a_3 \times a_4 \times z \times \Delta T_{ci} + z \times 0.5^{a_3} + (1-z)^{a_3}\right]^{1/a_3} \tag{7}$$

$$z = \begin{cases} 1 & if \quad T_{ci} \leq (50^{a_3} - 0.5^{a_3})/(a_4 \times a_3) \\ -1 & otherwise \end{cases} \tag{8}$$

where C_t is the total cracked pavement area in year t ($m^2/100m^2$); *z* is an auxiliary variable (years); ΔT_{ci} is the time since cracking initiation (years); a_3, a_4 are model constants, which differ for all and wide cracking.

The HDM-III cracking prediction model was considered to have a limited application range due to the characteristics of cracking data used in model development [13]. The road sections in Brazil where cracking data was collected presented mostly fatigue related cracking with rare longitudinal and transversal cracking. Wet non-freezing climate conditions support the inexistence of thermal cracking. Also, reflection cracking situations were not separated from all cracking data collected. This issue affects the accuracy of cracking initiation predictions.

The current HDM model, version 4, was developed to overcome the previously cited problems of HDM-III, namely the consideration of different cracking mechanisms with a specific model for each. Three cracking types are distinguished [13]: structural (fatigue/load); thermal; reflection cracking. The total area of cracking in each year is equal to the sum of the three cracking areas. Structural and reflection cracking follows the two-phase modelling principle, with the time to cracking initiation given by:

$$T_{ci}^s = a_0 \times e^{(a_1 \times N80Y/SNC^2)} \tag{9}$$

$$T_{ci}^r = a_0 + a_1 \times H_{new} \tag{10}$$

where T_{ci}^s is the time to structural cracking initiation (years) (C_t = 0.5%); *N*80*Y* is the indicative annual 80 kN ESAL (million ESAL/lane); *SNC* is the modified structural number for the pavement; T_{ci}^r is the time to reflection cracking initiation (years); H_{new} is the thickness of the new surface layer (mm); a_0, a_1 are model constants (defined for each mode). Cracking progression is estimated as follows:

$$CS_t = (1-z)\times 50 + z\times\left[\left(a_2\times SNC^{a_3}\right)\times a_4\times z\times N80_{ci} + z\times 0.5^{a_4} + (1-z)^{a_4}\right]^{1/a_4} \quad (11)$$

$$z = \begin{cases} 1 \; if \; T_{ci} \leq (50^{a_4} - 0.5^{a_4})/(a_3 \times a_4) \\ -1 \; otherwise \end{cases} \quad (12)$$

$$\Delta CR_t = \frac{a_2}{max(H_{new}, 25) + a_3} \times 100 \quad (13)$$

where CS_t is the total (structural) cracked pavement area in year t ($m^2/100m^2$); z is an auxiliary variable; $N80ci$ is the cumulative 80 kN ESAL at age Tci (million ESAL/lane); ΔCR_t is the change in reflection cracking area in year t ($m^2/100m^2$); a_2, a_3, a_4 are model constants (a_2, a_3 defined for each mode).

For thermal cracking, the NLDI study [13] states that it initiates immediately after construction, as due to the materials characteristics and environmental conditions, and only progression is modelled. However, in a more recent study [24], a different model was adopted that follows the two-phase principle:

$$T_{ci}^{t} = max[1, CDS \times CCT] \quad (14)$$

$$\Delta CT_t = \frac{\Delta NCT_t}{20} = \frac{1/CDS \times max\left\{0, min\left[NCT_{eq} - NCT_a, \left(2\times NCT_{eq}\times\left(age - T_{ci}^{t} - 0.5\right)/T_{eq}^{2}\right)\right]\right\}}{20} \quad (15)$$

where T_{ci}^{t} is the time to thermal cracking initiation (years); CDS is the construction defects indicator; CCT is the coefficient of thermal cracking; ΔCT_t is the change in the area of thermal cracking during the year t ($m^2/100m^2$); ΔNCT_t is the change in the number of thermal cracks during the year t (nº/km/lane); NCT_{eq} is the number of transversal cracks at equilibrium; T_{eq} is the time until equilibrium is reached (years); NCT_a is the number of transversal cracks at the start of analysis year.

PARIS Models

The PARIS project (Performance Analysis of Road Infrastructure) was a European research project carried out by members of the Forum of European National Highway Research Laboratories (FEHRL) during the 1990's aimed at the development of "a coherent set of pavement deterioration models" [18]. The development of the models was based on distress data (960 test sections) from the participant road network (real-time load testing) and some accelerated load testing experiments (France, Spain and Switzerland). For cracking, a distress initiation model and a distress propagation model were considered while for the other distresses (rutting, ravelling and roughness) only propagation models were proposed.

The pavement condition data collected in the different countries was normalized in terms of the severity and extent of distress. Regarding cracking, the inspection guidelines established three categories (longitudinal, transversal and alligator), two locations (in and out of the wheel-path) and three levels of severity (low, moderate and high), and the extent of distress defined by the linear length of test section affected. Cracking initiation was defined as the first appearance of longitudinal, transversal or alligator cracking, with a minimum of 0.5 m. The model proposed for the prediction of cracking initiation in flexible pavements is:

$$N100_{ci} = 10^{a_0 - a_1\times SCI_{300} - a_2/(SCI_{300}\times N100Y)} \quad (16)$$

where $N100_{ci}$ is the cumulative traffic load (100 kN ESAL) at the cracking initiation (years); SCI_{300} is the Surface Curvature Index 300 (μm), defined as the difference between the central deflection under the load and at 300 mm measured with a Falling Weight Deflectometer (FWD) applying a 50 kN load (pavement reference temperature 20 ºC); $N100Y$ is the average annual cumulative traffic load (100 kN ESAL); a_0, a_1, a_2 are model constants. This model is similar to the HDM models as it includes the same two explanatory variables, traffic and bearing capacity, with a similar mathematical expression.

For the development of the cracking progression model, cracking was normalized to the Cracking Index, CI, as bellow:

$$CI = 2 \times AC + LC + TC \quad (17)$$

$$AC = AC_{low} + 1.5 \times AC_{moderate} + 2 \times AC_{high} \quad (18)$$

$$LC = LC_{low} + 1.5 \times LC_{moderate} + 2 \times LC_{high} \quad (19)$$

$$TC = TC_{low} + 1.5 \times TC_{moderate} + 2 \times TC_{high} \quad (20)$$

where AC is the alligator cracking normalized extension (m); LC is the longitudinal cracking normalized extension (m); TC is the transversal cracking normalized extension (m).

Two different models were presented for the prediction of cracking progression in flexible pavements, with a single explanatory variable, either the traffic or the time, as follows:

$$log_{10}\left[\left(\Delta C/\Delta N100\right)_{fut}\right] = a_0 - a_1 \times log_{10}\left[\left(\Delta C/\Delta N100\right)_{old}\right] \quad (21)$$

$$\left(\Delta C/\Delta t\right)_{fut} = a_0 - a_1 \times \left(\Delta C/\Delta t\right)_{old} \quad (22)$$

where $\left(\Delta C/\Delta N100\right)_{fut}$ is the cracking index slope with the cumulative traffic load (100 kN ESAL) in the future (%/100kN ESAL); $\left(\Delta C/\Delta N100\right)_{old}$ is the cracking index slope with the cumulative traffic load (100 kN ESAL) in the past (%/100kN ESAL); $\left(\Delta C/\Delta t\right)_{fut}$ is the cracking index slope with the time in the future (%/year); $\left(\Delta C/\Delta t\right)_{old}$ is the cracking index slope with the time in the past (%/year); a_0, a_1 are model constants (defined for each model). None of the pavement construction variables or the climatic variables were found to correlate well with cracking progression. The two expressions are linear models and do not fit data well beyond cracking levels of 80%. Nevertheless, such high levels of pavement degradation are hardly ever found in practice because M&R actions are taken before such levels are reached.

The HDM and PARIS models are solid pavement performance models developed at a transnational level with pavement data from different regions and varied road conditions. HDM models benefit from the fact that they were adopted by many road administrations worldwide for their PMS. On the contrary, PARIS models were developed by a cooperation of European researchers but no road agency adopted the proposed models.

HDM models are absolute-type models for use at both the network- and the project-level. Thus, they are a key tool in the technical-economical evaluation of road projects financed by the World Bank. Differently, cracking progression was modelled in PARIS project with a relative-type model. This model was developed for use at the network-level. However, cracking evolution cannot be predicted on the long-term because cracking progression rates change with time and cracking level.

It is the aim of this study to adapt these models for Portuguese conditions, allowing its use in the Portuguese PMS.

Cracking Prediction Model Development

The development of the cracking prediction model for the Portuguese road network was primarily defined to be based on the HDM models and on the PARIS models. HDM models are used worldwide, however, the simple determination of the local calibration factors (*K*) was considered not feasible due to the differences in the procedures used to assess pavement cracking in Portugal. PARIS models were selected because they were developed from European road pavement condition data.

The methodology used in this study was to adjust the models expressions to the database and in the cases where the adjustment is weak, to propose the change of the variables in the expressions. First, the selection of data is presented and the considerations taken for the analysis. The development of a model is described for predict cracking initiation and following that, is the model for the cracking progression.

Data Selection and Preparation

For the development of the pavement distress evolution model, a sample of sections (45) from the database was selected with the following considerations:
- spatial distribution by the country;
- variation of traffic levels;
- variation of climatic conditions;
- variation of pavement structural capacity;
- included information.

About the climatic conditions variation in Portugal, excluding the Madeira and Azores islands, Fig. 3 shows the variation in the country of average temperature and annual precipitation [25]. The territory was divided into zones and plotted according to annual average precipitation and temperature values, each with 3 different classes as follows: for the temperature, the classes A (< 13℃), B (13-16℃) and C (> 16℃), and for the precipitation, the classes D (>1600 mm/year), E (600-1600 mm/year) and F(< 600 mm/year).

The manual for pavement condition inspection [16] identifies three cracking classes, based on type and severity, and establishes the quantification of the extent of cracking from the measurement of the road length affected by each cracking type. The cracking condition of the section is assessed with the cracking area (C):

$$C = (0.5 \times C_1 + 2.0 \times C_2 + W \times C_3)/(L \times W) \qquad (23)$$

where C_1 is the length of level 1 cracking (m); C_2 is the length of level 2 cracking (m); C_3 is the length of level 3 cracking (m); L is the length of section (m); W is the width of lane.

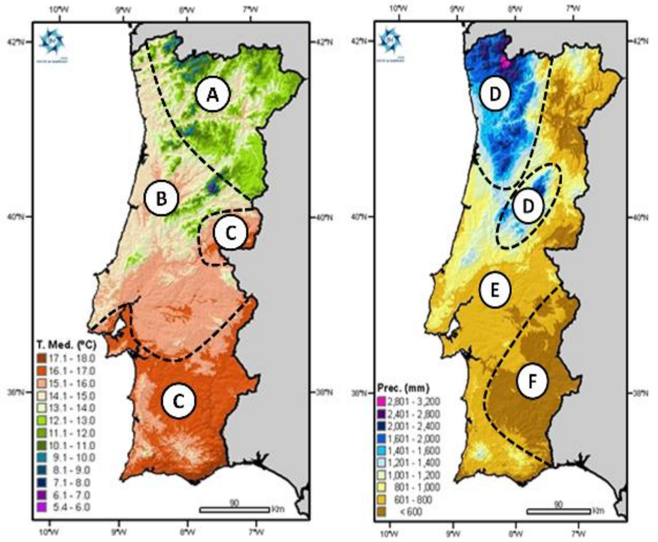

Fig. 3. Climatic Conditions in Portugal: Average Temperature (Left); Average Annual Precipitation (Right).

The data used in this study was collected in the 2003, 2007 and 2010 campaigns, and based on visual inspection made by trained technicians. Cracking extent is expressed as area (or percentage of area) using normalized width values (0.5 m for line cracks and 2.0 m for open and/or interconnected cracks). Fig. 4 shows a flowchart that illustrates the procedure taken to select the test sections for the development of the cracking prediction model.

The pavement structural capacity is characterized in EP-PMS with the AASHTO structural number SN while in HDM models the modified version SNC is used. Structural and drainage factors are constant irrespective of the climatic conditions variations in the country. SNC was determined, using Eqs. (5) - (6), with both factors redefined to meet the conditions found in Portugal. Regarding drainage, it was considered that the drainage factor varies with the average annual precipitation value as following, 1.0 for moderate precipitation values (zone E), 0.85 for higher than average precipitation values (zone D) and 1.15 for less than average precipitation values.

For the definition of the structural factors, the materials characteristics and the climatic conditions (temperature) were considered. The AASHTO manual [23] proposes the following structural coefficients (SN/inch): 0.200-0.425 for asphalt surface courses; 0.100-0.205 for cement base courses; 0.120-0.240 for asphalt base courses; 0.040-0.140 for granular base courses. Considering that continuously graded asphalt mixtures with non-modified bitumens (35/50 or 50/70 grade) are commonly used, the asphalt layer stiffness modulus was assumed to be 4000, 4500 and 5500 MPa for the zones A, B and C respectively. The structural coefficient of two different layers is related to the layers' stiffness modulus [26], as:

$$C_1^e / C_2^e = \left(\mathrm{E}_1 / \mathrm{E}_2\right)^{1/3} \qquad (24)$$

where C_1^e and C_2^e are the structural coefficient of layer 1 and 2, respectively; E_1 and E_2 are the stiffness modulus of layer 1 and 2,

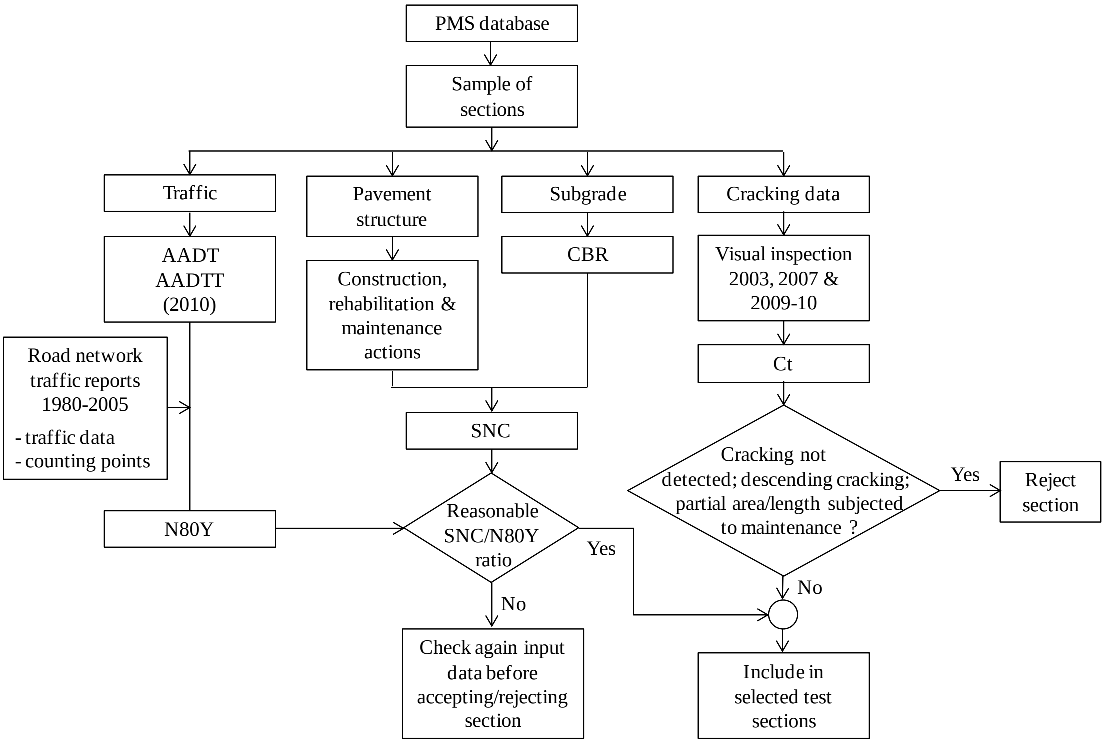

Fig. 4. Flowchart of Test Sections Selection.

Table 2. Variation of Structural and Drainage Coefficients with the Location and the Materials Used.

Materials	Structural Coefficients			Drainage Coefficients		
	Zone A	Zone B	Zone C	Zone D	Zone E	Zone F
(hot-mix) Asphalt	0.53	0.50	0.48	0.85	1.0	1.1
(hot-mix) Asphalt (Cracked)	0.14	0.14	0.14	0.85	1.0	1.1
Semi-penetration Macadam	0.2	0.2	0.2	0.85	1.0	1.1
Well-graded Crushed Aggregate &Hydraulic Macadam	0.14	0.14	0.14	0.85	1.0	1.1

respectively. Therefore, as 0.425 is for an asphalt surface course with a stiffness modulus of 2800 MPa [23], at 20℃, the values of 0.480, 0.500 and 0.530 were determined, with Eq. (24), for the zones A, B and C respectively. Nevertheless, these values are only valid in absence of cracking. As the cracking extent increases the ability of the layer to spread loads is highly diminished. The AASHTO manual proposes decreasing values of the structural coefficient as a function of the extent and severity of alligator and transversal cracking, which may be as low as 0.080. Hence, it was considered that more than 10% of the area has high severity alligator and transversal cracking in case of a cracked asphalt layer. Table 2 shows the structural and drainage coefficients for all the combinations considered.

For the use of the PARIS model, the SCI_{300} is required. This deflection parameter is obtained from FWD tests that were not available in database. So, SCI_{300} values were estimated via the mechanistic analysis of the FWD test on the different pavement structures, using Elsym5 software.

The traffic information available for each segment was verified against the national traffic statistics available from 1980 to 2005. The average daily truck traffic was converted to 80 and 100 kN ESAL, using the equivalency factors (Table 3) considered in the Portuguese pavements design guide –MACOPAV [27].

Table 3. Equivalent Standard Axle Load Factors (80 and 100 kN) according To the Traffic Class.

Truck Traffic (Daily)	Load Factor	
	80 kN	100 kN
50-150	2.00	0.82
150-300	3.00	1.23
300-500	4.00	1.64
500-800	4.50	1.84
800-1200	5.00	2.05
1200-2000	5.50	2.25

Fig. 5 shows the SNC-N80Y relation of the sections for the cracking model development. As expected, SNC increases as the traffic load intensifies. However, for the same traffic level there are pavements with substantial differences in bearing capacity. Very high or very low SNC/N80Y ratio values indicate sections with possibly erroneous traffic/pavement data or incorrect pavement design (under- or over-design). For the sections falling in these situations, the data was carefully reviewed before deciding on accepting or eliminating the section from the modelling sample.

Cracking Initiation

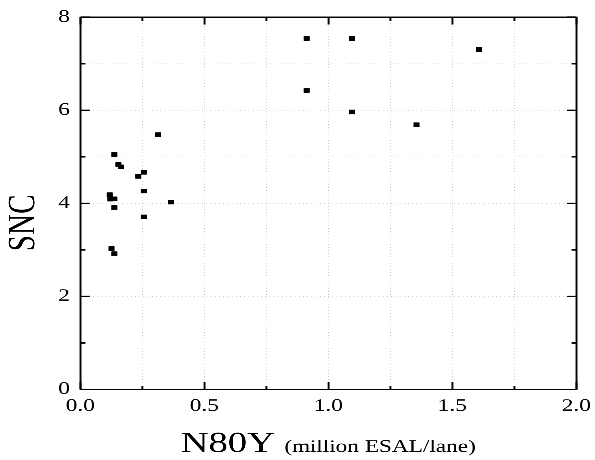

Fig. 5. SNC vs N80Y in Sample.

The initiation of cracking was modelled using the same definition as in the HDM models (time to 0.5% of cracked area). The point of cracking initiation was determined from the intercept of a linear progression line made for the two lowest cracking values.

Fig. 6 compares predicted and field values of cracking initiation from adjustment to data of Eqs. (4) and (9), considering the time to cracking initiation (HDM models), and of Eq. (16), considering the cumulative traffic load to cracking initiation (PARIS model). The adjustment results are very poor for both HDM models, and very good with the PARIS model (R^2 = 0.92).

Considering the inadequacy of HDM models to capture the field behaviour, the cracking initiation point was redefined from time to traffic basis. Thus, Eqs. (4) and (9) are replaced by

$$N80_{ci} = K_{ci} \times \left[a_0 \times e^{(a_1 \times SNC + a_2 \times YE4/SNC^2)} + CRT\right] \quad (25)$$

and

$$N80^{s}_{ci} = a_0 \times e^{(a_1 \times YE4/SNC^2)} \quad (26)$$

where $N80ci$ is the cumulative 80 kN ESAL at the time of cracking initiation (million ESAL/lane); $N80^{s}_{ci}$ *is* the cumulative 80 kN ESAL at the time of structural cracking initiation (million ESAL/lane). Fig. 7 compares field and predicted values of the cumulative traffic for cracking initiation. Both models can capture the cracking initiation behaviour of real pavements, where the adjustment to data of modified HDM-III model is as good as with the PARIS model.

Cracking Progression

The progression of cracking is modelled from the PARIS and HDM-III cracking progression models, and from the HDM-4 model for the structural cracking evolution. There are two reasons for developing a model for the progression of all cracking. First, cracking condition in EP-PMS is expressed as the area (or percentage of) affected by cracking, estimated with Eq. (23), without the identification of the mechanism that originated it. Second, the thermal contribution for cracking (transversal) can be neglected due to dry-summer subtropical climate in Portugal [24, 25]. Nevertheless, it is considered important in the future to separate

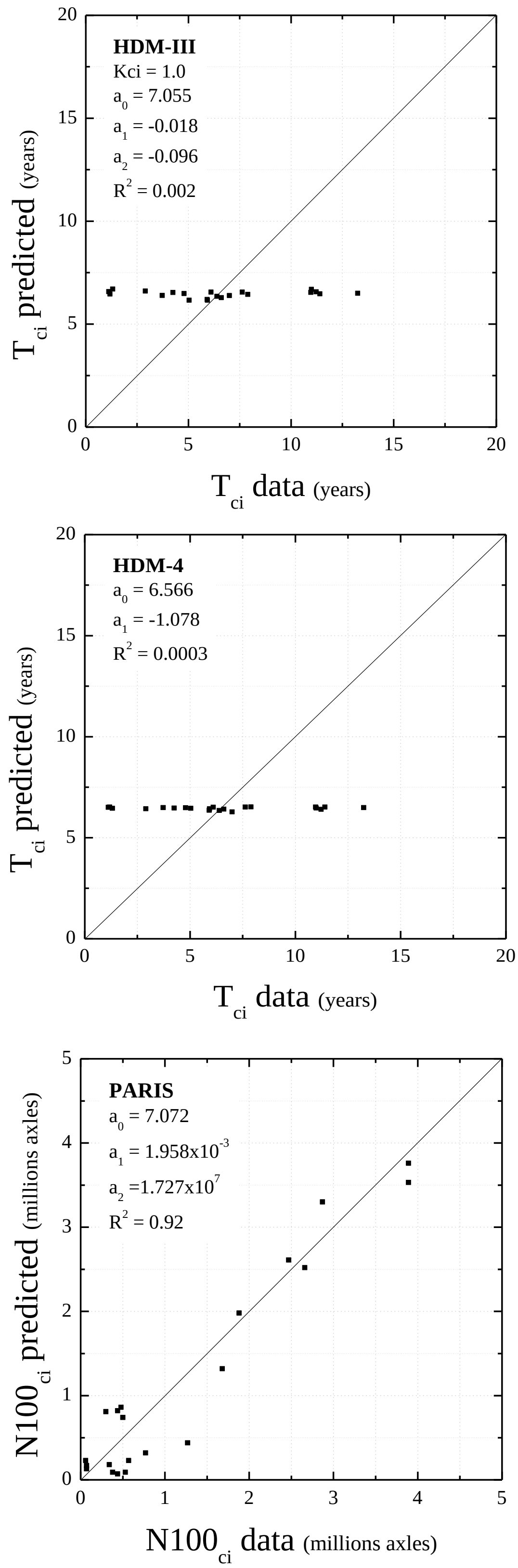

Fig. 6. Cracking Initiation Values vs Predicted with Adjusted Models (Original).

the fatigue/structural cracking that evolves in pavement from construction from the situations of reflection cracking, either from overlay on top of cracked pavement or from semi-rigid pavements.

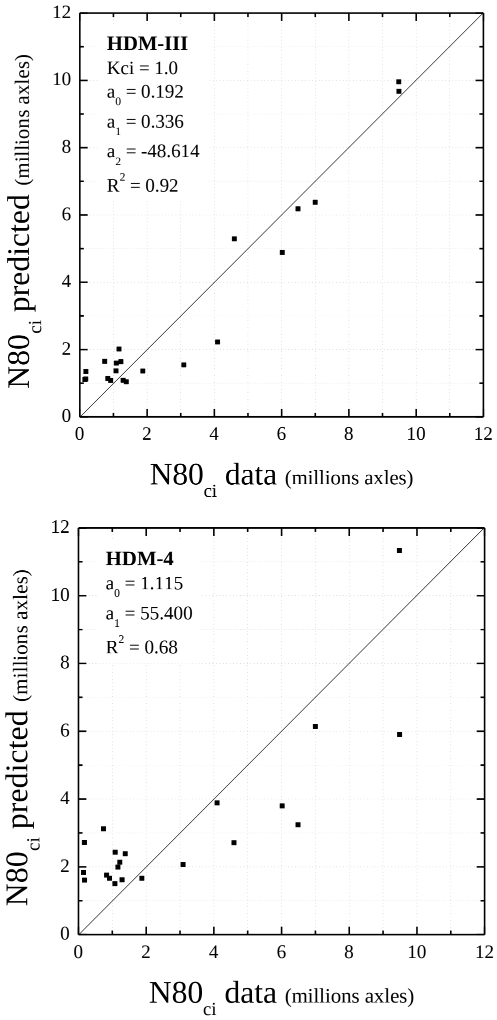

Fig. 7. Cracking Initiation Values vs Predicted with Adjusted Models (Modified).

Fig. 8 compares predicted and field values of cracking progression from adjustment to data of Eqs. (7) and (11), considering the area of cracked pavement (HDM models), and of Eq. (21), considering the cracking progression slope as a function of cumulative traffic load (PARIS model).

The PARIS model with the cracking progression slope as a function of the time was not considered due to the previous inadequacy of the models with the time variable for the cracking initiation prediction. The results showed that the model developed based in the PARIS study is able to predict very well the cracking evolution, with a correlation of R^2 = 0.89. In the case of the HDM-III, this model explains well the tendency of progression (R^2 = 0.68), while the HDM-4 has a poor adjustment to data (R^2 = 0.39). As the values of cracking area in the sample do not exceed 50% it was not possible to fit the entire S-curve of the HDM models. The substitution of variables in HDM models, either time to traffic or the opposite, did not improve the adjustment to field data significantly.

The superior adjustment of the PARIS model is likely to be related to the fact that it is not a model for the prediction of the extent of cracking at a certain time point in the pavement life but a model that predicts distress growth in the next time period using

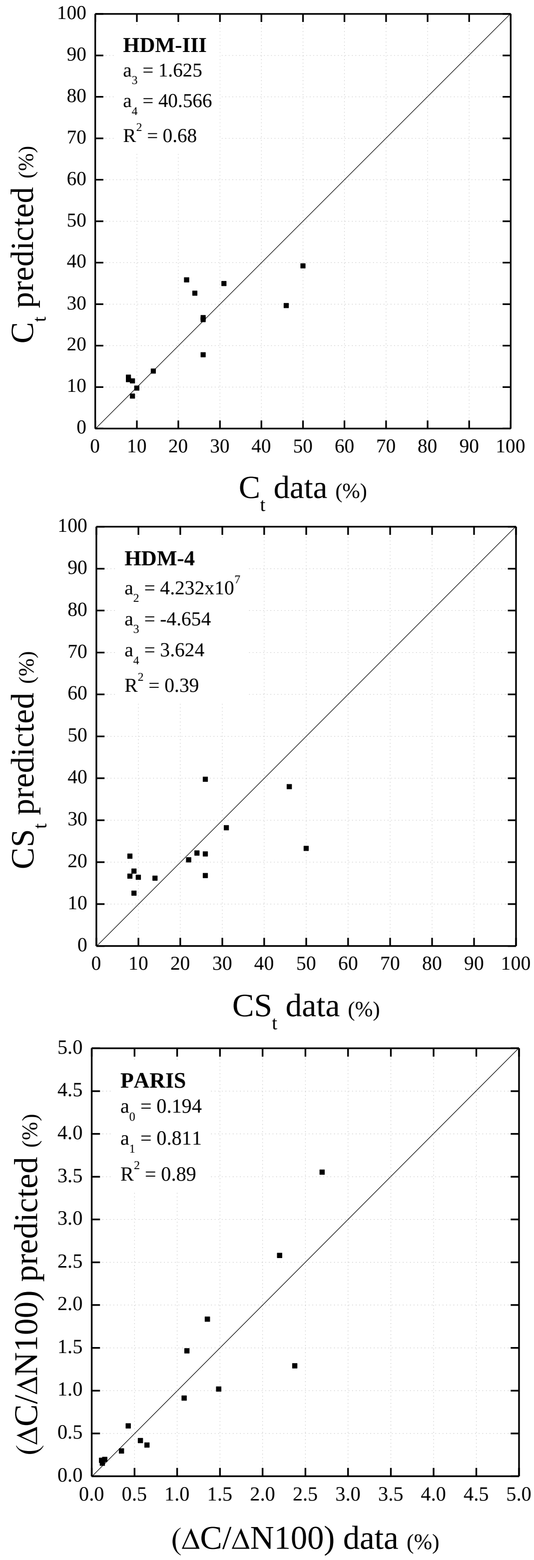

Fig. 8. Cracking ProgressionValues vs Predicted with Adjusted Models.

information from the immediately previous period. It is a relative model, which is suitable for pavement management purposes at the short- and medium-term while it is not useful for long-term analysis and for the comparative study of alternative pavement structures at the project-level.

Recommended Model

Description of the Model

From the previously presented results, it is proposed the following model to be implemented in EP-PMS:

$$N100_{ci} = 10^{7.072-1.953\times10^{-3}\times SCI_{300}-1.727\times10^{7}/(SCI_{300}\times N100Y)} \quad (27a)$$

$$log_{10}\left[\left({}^{\Delta C}/_{\Delta N100}\right)_{fut}\right] = 0.194 - 0.811 \times log_{10}\left[\left({}^{\Delta C}/_{\Delta N100}\right)_{old}\right] \quad (27b)$$

$$C_t = (65.920 \times \Delta T_{ci} + 0.5^{1.625})^{1/1.625} \quad (27c)$$

where C_t is the cracked area in year t (m^2/m^2); ΔT_{ci} is the time since cracking initiation (years); and all other variables are as referred previously.

Cracking initiation is predicted with the PARIS project based expression, Eq. (27(a)). The obtained constants for the Portuguese conditions are similar to the PARIS project ones (Eq. (27(a)): 7.287, 6.7×10^{-3}, 2.28×10^{6}; Eq. (27(b)): 0.29, 0.96) which were determined using data from different European countries (Denmark, Finland, France, Great Britain, Greece, the Netherlands, Sweden, Switzerland) [18].

For the cracking progression prediction two different situations are considered: (i) on short- to medium-term is recommended a relative model, Eq. (27(b)); and (ii) on the long-term, the absolute model given by Eq. (27(c)). Due to the limited range of cracking area values in the sample used for the model development, the absolute model predictions are only valid for cracking values up to 50%. Eq. (27(c)) is obtained from the simplification of Eq. (7) by adopting only the first half of the S-shaped function.

Traffic has a high influence on the cracking initiation phenomenon while for progression it depends on the model considered. Instead of time to cracking initiation the accumulated traffic load to cracking initiation is adopted. The use of the traffic variable is methodologically positive as it applies the same variable used in pavement design procedures and in the pavement management system it can be interpreted as the already spent part of the pavement life. Also, SCI_{300}, an index calculated from FWD deflection measurements, which substitutes SNC, is adopted. Although deflection data is not available in most road sections it can be easily collected and integrated into the medium- to long-term pavement evaluation plans. Likewise, Bryce *et al.* [28] concluded that when using a structural condition measure, network-level pavement management decisions are closer to the those taken at the project-level than when only surface distress condition is considered, and also recommends using it to improve currently used pavement deterioration models. The structural evaluation with FWD is already included in final inspection plans performed before work hand-over on major rehabilitations and new constructions. Thus, SNC has never been calibrated for the Portuguese conditions. Nevertheless, for an easier integration of the model in the current PMS an expression relating SNC and SCI_{300} was determined. In Fig. 9, SNC versus SCI_{300} is plotted for the sample used in this study and the expression obtained for the relation between the two variables is presented.

Regarding the models proposed for cracking progression prediction, the absolute model is a time-function while the relative model uses past distress growth rate. Both models are proposed to be implemented in PMS with different goals. The absolute model is selected for the road network's M&R programming when considering long time spans (10 years or more), using the implemented strategies evaluation tool. Jorge and Ferreira [12] describe the modification of the current SET in EP-PMS to integrate into the optimization model the use of pavement performance models for each distress type.

On the other hand, for the M&R programming over short to medium-term, up to 5 years, and to guarantee that the limit on cracking values defined in the QCP is not exceeded in any road segment, the relative model is recommended, which uses past data of each segment and has shown very good adjustment to data.

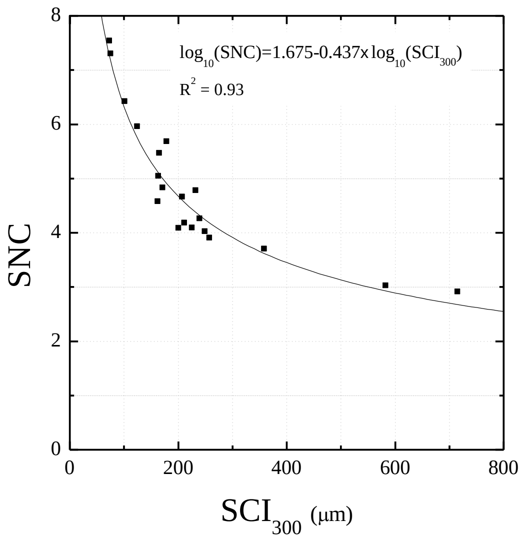

Fig. 9. SNC vs SCI_{300}.

Application of the model to Portuguese Pavement Structures

In this section, the proposed model was used at the project-level to determine cracking progression in some pavement structures. The time to cracking initiation and the time to intervention with M&R actions (cracking warning level of 20%) were predicted, using the absolute model version, for 9 pavement structures designed with the Portuguese pavement design manual (MACOPAV) [27]. The MACOPAV considers a total of 16 different pavement structures that result from the combination of 6 traffic classes and 4 foundation classes.

Table 4 describes the 9 pavement structures analysed, considering a pavement structure with asphalt surface and base layers on a granular sub-base layer. These structures were obtained from combination of three traffic classes (T5: 300 trucks/day; T3: 800 trucks/day; T1: 2000 trucks/day) with three subgrade classes (F2: CBR = 10%, E = 60 MPa; F3: CBR = 20%, E = 100 MPa; F4: CBR = 30%, E = 150 MPa).

Fig. 10 illustrates the cracking evolution of the 9 pavements predicted with the model, Eq. (27(a)) and (27(c)). The results show a considerable dependence of the traffic level, especially from T3 to T1, though the pavement structure is thicker when a higher traffic

Table 4. Characteristics of Pavement Structures.

Pavement Structure	Surface Layer (Asphalt Concrete)			Base Layer (Asphalt Concrete)			Sub-base Layer (Granular)		
	H (mm)	E (MPa)	ν	H (mm)	E (MPa)	ν	H (mm)	E (MPa)	N
P1	40	4000	0.35	60	4000	0.35	200	200	0.35
P2	40	4000	0.35	80	4000	0.35	200	200	0.35
P3	40	4000	0.35	120	4000	0.35	200	200	0.35
P4	40	4000	0.35	140	4000	0.35	200	200	0.35
P5	50	4000	0.35	140	4000	0.35	200	200	0.35
P6	50	4000	0.35	160	4000	0.35	200	200	0.35
P7	40	4000	0.35	180	4000	0.35	200	200	0.35
P8	50	4000	0.35	170	4000	0.35	200	200	0.35
P9	50	4000	0.35	190	4000	0.35	200	200	0.35
P10	60	4000	0.35	180	4000	0.35	200	200	0.35
P11	50	4000	0.35	200	4000	0.35	200	200	0.35
P12	60	4000	0.35	200	4000	0.35	200	200	0.35
P13	50	4000	0.35	230	4000	0.35	200	200	0.35
P14	60	4000	0.35	220	4000	0.35	200	200	0.35
P15	60	4000	0.35	240	4000	0.35	200	200	0.35
P16	60	4000	0.35	260	4000	0.35	200	200	0.35

Notes: H – layer thickness; E – stiffness modulus; ν – Poisson's ratio.

level is considered. For the T5 class, cracking occurs after 7 years while it initiates before the 4th year for the T1 class pavements. The 20% of cracking area is attained in 2 years after cracking initiation in all situations. The pavement structures proposed for T1 class are underdesigned considering usual intervals for M&R actions, and when compared with T3 and T5 proposed structures. The effect of the subgrade class variation is small, being only detected in T5 results, which is due to the small variation of SCI_{300} values for the different pavement structures. The increase in asphalt thickness compensates the weaker subgrade and similar values for the SNC are obtained. Thus, the results indicate that a revision of the Portuguese pavement design manual [27] should be performed to attend the larger required bearing capacity of the pavement when a higher traffic intensity is considered.

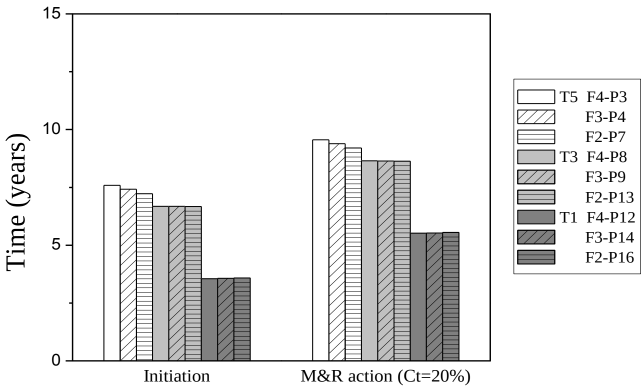

Fig. 10. Predicted Time to Cracking Initiation and to 20% of Cracking Area.

Conclusions

This paper presents the first part of an on-going research study aiming the development of pavement deterioration models adapted for the Portuguese road conditions. A flexible pavement cracking prediction model, with separation of initiation and progression phases, was developed with data from current EP-PMS database and supported on the HDM and PARIS study models.

Cracking initiation is better expressed in terms of accumulated traffic load instead of time elapsed, and determined from pavement structural capacity and annual traffic load. The SCI_{300} index is preferred instead of SNC for structural capacity description as it is obtained from field measurements. On the contrary, SNC has never been calibrated for local conditions. Nevertheless, a correlation between SCI_{300} index and SNC for the evaluated network was developed.

Cracking progression prediction with a relative model shows better results and it is recommended for short to medium-term analysis. For the optimization of the M&R plan over long periods (10 years for example), which is especially important for boards to define their company's financial plan, an alternative model for cracking progression prediction is proposed.

In the future, the intention is to implement the models in EP-PMS and validate it with upcoming data. Current surveys are done with an automatic collection system that increases the reliability of database information. The proposed model is a valuable tool for the Portuguese concessionaires dealing with the recent demanding Portuguese legislation.

Acknowledgments

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