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Generalized Linear Prediction Performance Model for Asphalt Overlays in a Heavy Traffic Urban Road

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Abstract: This paper presents the results of a study of the performance of asphalt overlays rehabilitation through milling and filling (replacing old asphalt surfaces) of heavily loaded urban roads in São Paulo City (Brazil). Pavement sections were monitored upon completion of the last rehabilitation. Field investigations conducted included measurements of roughness and cracking progression. One peculiar characteristic of the pavement sections is the severe hydraulic base contamination and deterioration. Based on measurements of structural and functional pavement performance parameters, empirical models were developed to predict the performance of roughness and cracking of asphalt overlays by generalized linear models. Comparisons of new prediction models with HDM-4 performance models for asphalt over granular bases were conducted. It was noted that the proposed cracking models gave more conservative prediction than the HDM-4 performance models provided. However, the derivate roughness models resulted in similar predictions as provided by the HDM-4 models.

***Key words**: HDM-4; Maintenance; Models; Pavements; Performance.*

Introduction

Most part of the urban expressways in São Paulo was built during the 1960s and 1970s linking several rural highways in city's neighborhoods. The typical pavements structure consists of an asphalt layer on top of hydraulic macadam bases over poor subgrade soils. As a result of increasing truck volumes during the last two decades, these pavements had shown poor performance. The traffic volumes are as high as twelve thousand trucks per day. Many factors had contributed to these poor performances, such as bad drainage condition, poor subgrade, and eroded hydraulic bases. After over three to four decades of heavy traffic, majority of the original macadam base layer has deteriorated and pumping was evident. Although several maintenance activities have been executed, they have not been successful in restoring the pavements. The high volume traffic fleeing on these pavements makes it very difficult for reconstruction works within the lanes. Severe grade restrictions do not allow normal construction operations, which might cause unacceptable traffic congestion. Therefore, milling and overlay, designated as "mill and fill" in São Paulo has been the only maintenance activity possible, which is normally performed at nights. However, mill and fill does not address the problems caused by poor subgrade and deteriorated base.

The objective of this study is to evaluate the performance of this type of urban pavements through extensive monitoring of one of the most important expressways within city limits. Performance data were collected between 2002 and 2005. Generalized linear models were employed to develop performance models of asphalt overlays (mill and fill) with conventional dense graded asphalt concrete mix. The developed performance models were then compared to similar models used by the Highway Development and Management (HDM-4). The main purpose of this research was to develop performance prediction models that can be used to predict performances of mill and fill overlay as pavement rehabilitation technique that has been broadly employed by the city road agency. The models can also be used as tools for scheduling and planning maintenance activities and budget for the coming years.

Backgrounds and Selection of Test Sections

The Bandeirantes Ave. was chosen for the study because of its poor condition indicating the need of "mill and fill" rehabilitation in the near future (less than five years). The pavement structure is typical of this region, consisting of asphalt surface with severe deteriorated hydraulic macadam base. Further, the high volume of trucks traveling on the road would normally cause the new pavement overlay to deteriorate rapidly, making it possible to analyze the pavement performance in a short period of time. This study can provide valuable information for management of many similar pavements across the city. The expressway under evaluation is 8km long, with 4 lanes in each direction and comprising sections of two typical soil foundations in Sao Paulo city geological plate: sand-silt alluvial transported soils and residual silt-clay expansive fine soils. Legal limits of truck axle loads in the country are 80, 170, and 255kN for single, dual tandem, and tridem axles, respectively.

The expressway roads are normally subjected to trucks with more than two axles. The most common types of trucks in the city are those with a single front axle and two rear axles, consisting of one single (twin wheel) axle and a tridem axle. Most of these urban

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Table 1. Location, Geometry, and Structure of the Selected Pavement Control Sections.

Traffic direction	Distance from origin (m)	Lane	Section	Alignment patterns: horizontal	Alignment patterns: vertical	Slope (%)	Thickness of layers (mm): Existing HMA	Granular base	Subgrade (HRB)	HMA Mill	HMA Replace
East	280	Right	S1	straight	level	-	550	150	A-7-5	50	50
	940		S2	straight	level	-	300	400	A-7-5	80	80
	1700		S3	straight	level	-	300	400	A-7-5	80	80
	3000		S5	straight	ascent	+2.8	210	300	A-7-6	150	150
	3100		S5b	straight	ascent	+4.2	210	300	A-7-6	150	150
	3200		S6	right	ascent	+5.0	220	200	A-7-6	220	220
	3300		S6b	straight	ascent	+6.3	220	200	A-7-6	220	220
	280	Central right	S1	straight	level	-	550	150	A-7-5	50	50
	940		S2	straight	level	-	300	400	A-7-5	150	150
	1700		S3	straight	level	-	300	400	A-7-5	150	150
	3000		S5	straight	ascent	+2.8	210	300	A-7-6	100	100
	3100		S5b	straight	ascent	+4.2	210	300	A-7-6	150	150
	3200		S6	right	ascent	+5.0	220	200	A-7-6	150	150
	3300		S6b	straight	ascent	+6.3	220	200	A-7-6	150	150
West	2680	Right	S7	straight	descent	-2.3	280	300	A-7-5	150	150
	2480		S7B	left	descent	-2.3	290	150	A-7-5	50	50
	2380		S7Bb	left	descent	-1.6	290	150	A-7-5	50	50
	2280		S7Bc	left	descent	-1.6	290	150	A-7-5	50	50
	2180		S7Bd	left	descent	-1.6	290	150	A-7-5	50	50
	1380		S8	straight	level	-	290	150	A-7-5	50	50
	1100		S9	straight	level	-	280	300	A-7-5	50	50
	600		S10	straight	level	-	280	300	A-7-5	50	50
	2680	Central right	S7	straight	descent	-2.3	280	200	A-7-5	50	50
	2480		S7B	left	descent	-2.3	290	150	A-7-5	50	50
	2380		S7Bb	left	descent	-1.6	290	150	A-7-5	50	50
	2280		S7Bc	left	descent	-1.6	290	150	A-7-5	50	50
	2180		S7Bd	left	descent	-1.6	290	150	A-7-5	50	50
	1380		S8	straight	level	-	290	150	A-7-5	100	100
	1100		S9	straight	level	-	280	300	A-7-5	100	100
	600		S10	straight	level	-	280	300	A-7-5	50	50

roads were constructed during the sixties.

The last "mill and fill" maintenance work on this pavement was done in 2002 with overlay thickness from 5 to 22cm. The pavement test sections to be monitored in this study were selected based on their performance history, type of subgrade soil, and overlay thickness. Note that hydraulic macadam base erosion is a common characteristic for all test sections and all the expressway pavements which represent a typical condition for all urban roads in the city. The selection of the test sections was also aided by detailed inspections on the roadway pavements conducted before and during the 1992 and 2002 rehabilitation works. Cores were drilled and trenches were excavated along the truck lanes in the avenue. Thicknesses of the pavement sections were measured to ensure the homogeneity of each chosen test section. As shown in Table 1, thirty pavement sections were selected and systematically monitored since 2002, including fourteen sections in the eastbound lanes and sixteen in the westbound lanes. However, during field evaluation, some of the sections were found to have old concrete pavement underneath and were eliminated from this study with 24 pavement test sections left. All test sections are located in the right and the central right lanes (the expressway has four lanes in each direction). The dimensions of each test section are 100m long and 3.7m wide (the lane width). Major characteristics of the test sections are presented in Table 1, including both vertical and horizontal alignment patterns, existing pavement structures, subgrade soil classification, and thickness of the "mill and fill" operation. From Table 1, it is noted that soils for all test sections are classified as either A-7-5 or A-7-6, indicating weak and expansive subgrade soils. Some sections have very thick asphalt layer resulted from successive overlays in the past.

One important issue on this study is the relationship between the thicknesses of the new overlay and the existing hot mix asphalt (HMA). From Table 1, two main groups of existing pavement structures are noticed. For test sections on the eastbound lanes, a HMA layer about 200mm thick over a base layer of 200 to 400mm generally prevails those on the westbound lanes commonly with a HMA of 300mm thick over a thin (150mm) macadam base. The above ranges for both HMA and base thicknesses are selected for the test sections.

The thick bases are very common in the city's road system and

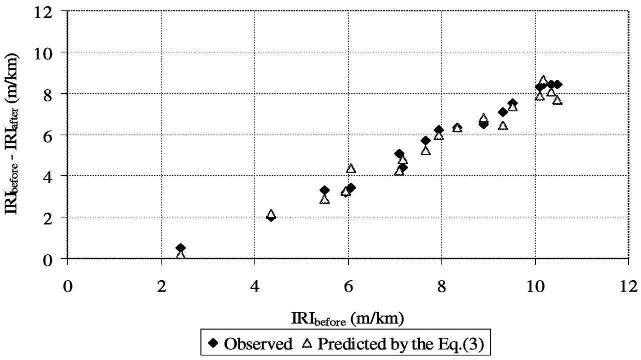


Fig. 1. Roughness Reduction Caused by the Mill and Fill Procedure for Pavement Overlaying.

are results of past construction activities. The original roadways were only two-lane streets. Prior to fifties and sixties, city street pavements were generally constructed with a thin sheet asphalt layer over macadam bases. The original granular bases were often kept as subbase during subsequent construction of the city roadways after the initial construction, as was the case of the eastbound lanes of the Bandeirante's Avenue. As a result of this rehabilitation practice, several recently built pavements over old streets in the city showed poor performance, most likely due to poor compaction on the existing granular subbases. Similar deflections were measured before and after the milling, which indicated that structural capacity provided by the remaining cracked HMA thickness was irrelevant to a successful overlay performance.

From Table 1, it is also interesting to note that the thickness of the mill and fill operation varied differently from section to section, especially for sections on the eastbound lane. As mentioned earlier, detailed inspections were conducted in 2002 before this study. Theses inspections showed that the fatigue cracking not only existed in top surface layer (most recent HMA overlay), but also in underlying (older) asphalt layers, for many parts of the roadway pavements. Therefore, it was difficult to determine what the main factor lead to the poor pavement performance. It could be the fatigue damage in the top asphalt layer of a pavement with insufficient structural capacity which caused by high stresses and strains induced by heavy truck traffic. Moreover, reflective cracking propagated from HMA layers underneath the top layer which compounded by the heavy truck loads, might be another cause for early pavement failure. Unfortunately, it is very difficult to distinguish failures caused by these two factors. As a result, different thicknesses of mill and fill for different sections were required.

Because of the roadway conditions in the city, it was necessary to develop simple performance prediction models that could be used citywide to help make decision for pavement management. The development of simple statistical tools for local engineers to use in routine pavement evaluation was endorsed by the local government officials.

Description of Field Evaluation

Pavement surface roughnesses were measured immediately before and after the mill and fill operations of the test sections. Follow-up measurements were conducted at the intervals of two to five months to obtain a total of 12 field surveys. Bump Integrator was employed for all measurements. All roughness tests were performed at night to minimize the impact on traffic. A calibration equation was used for each testing velocity and then transforms these registered readings in the Quarter-car Index (*QI*) and in International Roughness Index (*IRI*) [1].

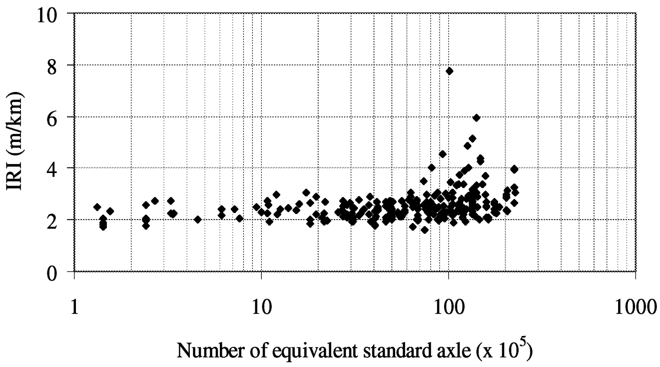
In order to monitor the types, quantities, and severities of the distress projects, pavement distress surveys were performed before the pavement overlay and at every four months after the mill and fill operation of the test sections. A total of 15 sets of pavement distress surveys were performed in this study. Structural assessment was carried out by using deflection data obtained from FWD deflections measurements prior and after overlay construction. The modified structural number (*SNC*) of the pavement structures was then determined.

Roughness Progression on Test Sections

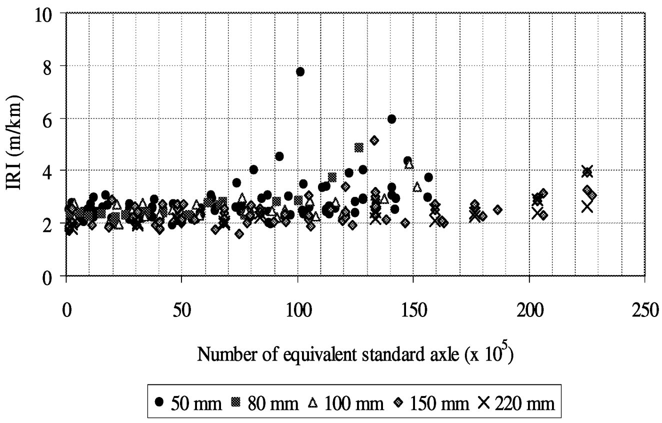
The "mill and fill" operations restored the riding quality of the pavements in 2002. Prior to the rehabilitation, the average pavement roughness values measured by *IRI* were 8.5 and 6.4m/km for the eastbound and westbound lanes, respectively. After the rehabilitation operations, the measured roughness values were 2.0 and 2.7m/km for the eastbound and westbound lanes, respectively. Note that, after the rehabilitation, the roughness values for most of the sections were about 2.0m/km which were irrespective of those values prior to the mill and fill rehabilitation. This observation agrees with the results obtained from HDM-4 models. Fig. 1 illustrates the linear relationship between the decrease in *IRI* after the rehabilitation and before mill and fill.

The relationship between measured roughness (*IRI*) and the number of accumulated equivalent single axle loads (ESALs) (herein denoted as *N*) for the pavement test sections was studied. It was noticed that *IRI* data for some sections on westbound lanes were significantly different from others. After further careful investigation, some sections were found to have high initial roughness right after mill and fill rehabilitation because of the poor construction. Moreover, some sections even had rigid base (old concrete pavement), which had significant influence on the measurement of the *IRI*. Hence, 24 sections were excluded from further evaluation.

Fig. 2(a) presents the roughness progression with traffic for all test sections. It can be seen that the relationship is not linear. The *IRI* stays essentially constant at 2m/km before the traffic reaches about 20 to 25 million ESALs. The measured *IRIs* then increase rapidly with the increasing traffic. Fig. 2(b) shows the progression of *IRI* for pavement sections with different "mill and fill" thickness. It is noticed that, by comparing the *IRI* at same traffic counts, the greater the overlay thickness the better the pavement performance. Sections with "mill and fill" thickness less than 100mm show a faster rate of *IRI* increase than those with larger "mill and fill" thickness. Moreover, an average *IRI* increase at about 0.5m/km was obtained from two successive surveys. It is also noted that most of the differences are on the positive side. For *N* close to about 50 millions ESALs, *IRI* values for sections with rehabilitation thickness less than 100mm are greater than those sections with

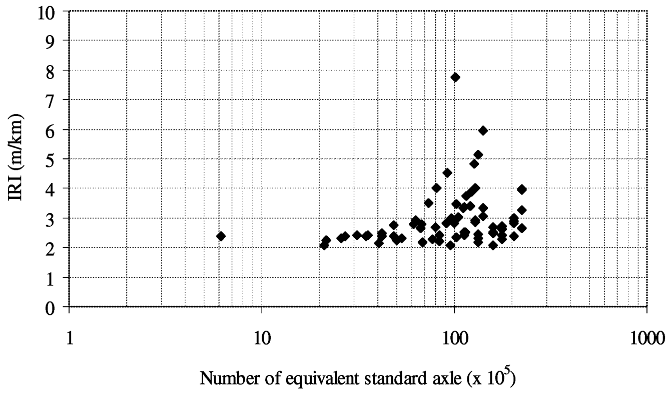


(a) For all sections

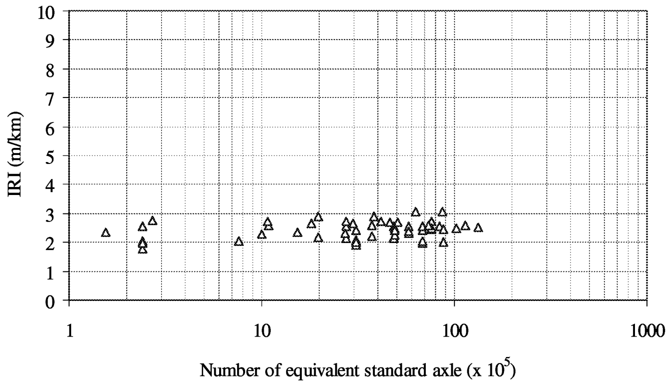


(b) For each overlay thickness value

Fig. 2. *IRI* Evolution as a Function of ESALs.



(a) Sections developing cracking patterns



(b) Sections without cracking

Fig. 3. Roughness Progression in Sections with and without Cracking.

thicker overlay.

It is generally recognized that structural conditions of the test sections could also influence smoothness of the pavement sections, *i.e.* the presence of cracking can accelerate the progression of the roughness for sections with different overlay thickness. However, no definite correlations were observed between the *IRI* values and the quantity of the cracked areas in the sections. The *IRI* values varied from 2 to 6m/km for a section with cracking area less than 5%. From Fig. 3, however, it is clear that the presence of cracking would increase the pavement roughness at higher values of *IRI*.

Cracking Progression Monitoring

Before the "mill and fill" rehabilitation, four test sections exhibited 100% cracked areas, and other sections presented 30 to 90% cracked pavement areas. Surface cracking was monitored starting from 2003 after overlays. Cracking was presented for four sections, S1, S2, S3, and S5b (all on the eastbound lane) at the first survey, with the cumulative ESALs less than 7 millions. Moreover, cracking started to develop for other test sections at cumulative ESALs over 20 millions. In particular, the distresses were not seen for sections S8 and S10 (both on westbound lanes) until the last survey conducted in 2005. Fig. 4 presents percentage of cracked areas as a function of traffic (cumulative ESAL) for the test sections. Sections with thin "mill and fill" overlays (50mm on the westbound lanes) gave the worst performance.

Development of Performance Models

Upon completion of the distress surveys, the performance models were constructed. In order to develop a generalized linear roughness and cracking performance models, variables and co-variables are defined in the models. The generalized linear model is shown below:

$$Y_n = X_{ik}\,\beta_i + W_j\,\gamma_j + \varepsilon \tag{1}$$

Where

Y_n = the response variables, with n = 1, 2, …, 5,

X_{ik} = the vector of co-variables with i = 1, 2…, 8 and k = 1, 2,…, 26,

- X_{1k} is related to roughness,
- X_{2k} is related to overlay thickness,
- X_{3k} is related to load repetition,
- X_{4k} is related to interaction of deflection, traffic and structural number,
- X_{5k} is related to age,
- X_{6k} is related to percentage of cracked area,
- X_{7k} is related to deflection,
- X_{8k} is related to structural number.

W = the vector of dummy variables, with j = 1,…, 4,

- W_1 is related to overlay thickness – *REF,*
- W_2 is dummy variable vector related to vertical

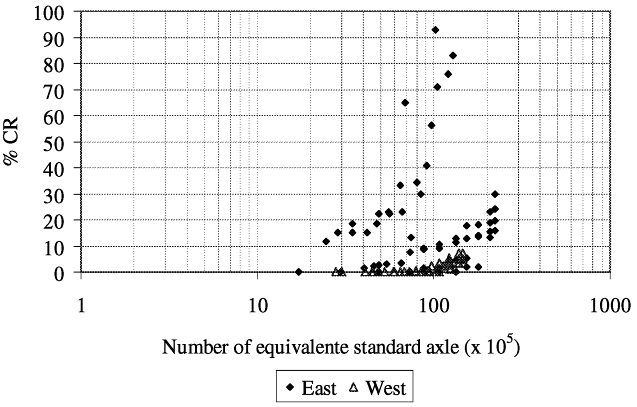


Fig. 4. Percentage of Cracking as Function of ESALs.

geometry – *GV,*
W_3 is related to soil type – *soil,*
W_4 is related to the presence or absence of cracking – *CR*.

β and γ = vectors of unknown parameters, with i = 1,…, 8 and j = 1,…, 4,
ε = the vector of random errors.

Note that transformations for dependent variables were used in building the performance models (see Table 2 for details). Observations obtained from different sections were assumed to be independent and ε followed a normal distribution, $\varepsilon \sim N$ (0, $\sum$), where $\sum$ is a positive symmetrical matrix. However, observations obtained from the same sections are not independent, since observations were taken at the same sections over a period of time, Hence, an autoregressive of order 1 covariance structure (AR1) was employed for the analysis by taking the following form:

$$\Sigma\,(\mathbf{Y}_n) = \sigma^2 \begin{vmatrix} 1 & \rho & \rho^2 & \cdots & \rho^{|T-1|} \\ \rho & 1 & \rho & \cdots & \rho^{|T-2|} \\ \rho^2 & \rho & 1 & \cdots & \rho^{|T-3|} \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ \rho^{|T-1|} & \rho^{|T-2|} & \rho^{|T-3|} & \cdots & 1 \end{vmatrix} \tag{2}$$

Estimation of the unknown parameters was obtained using the Procedure PROC Mixed of Statistical Analysis Systems (SAS) software, version 9. The residual analysis was applied to measure the fitness of the proposed models. The null hypotheses with the following form were tested in Eq. (1):

$$H_0\!: \beta_i = 0$$

$$H_0\!: \gamma_j = 0$$

Some structural components were adapted from models developed earlier in Brazil [2].

In order to screen the larger number of possible models, some strategies for the choice of the final models were adopted. A model would be selected as a candidate if it meets criteria defined as: all coefficients are statistically significant; the components of the model must satisfy limit conditions or physical principles related to the pavement deterioration; it must have at least one coefficient of determination greater than 0.3 (R^2>30%) and better values of Akaikes's Information Criterion (AIC) and Schwarz's Bayesian Criterion (BIC) [3]. By using such a strategy, appropriate models were selected by applying the statistical analysis.

Variation of the Roughness Due to Overlay (*ΔIRI*)

The model describing the correlation between the change in *IRI* (*ΔIRI*) and the thickness of overlay (mill and fill thickness) has the following form:

$$\Delta IRI = a + b \times TOV + c \times IRI_{before} \tag{3}$$

Where
ΔIRI = the decrease of roughness due to mill and fill (m/km),
TOV = the mill and overlay thickness (in cm),
IRI_{before} = the *IRI* before mill and fill (m/km),
a, *b*, and *c* = parameters of the model (see Table 3 for values).

Fig. 1 presents the observed values versus those predicted by Eq. (3) and good predictability is shown in the figure. The proposed model is similar to those obtained in literature for Brazilian conditions [4].

Roughness Progression (*IRI*)

Two models, Eqs. (4) and (5), were developed for predicting the *IRI* of the test sections after mill and fill. Coefficients of determination (R^2) are 31.9 and 54.3% for Eqs. (4) and (5), respectively. In these models, the *IRI* are functions of age, *N* (cumulative ESALs) and structural capacity which represented by deflection data.

$$IRI = \left(a + b \times REF - c \times D \times N - d \times \ln N\right)^{-1} \tag{4}$$

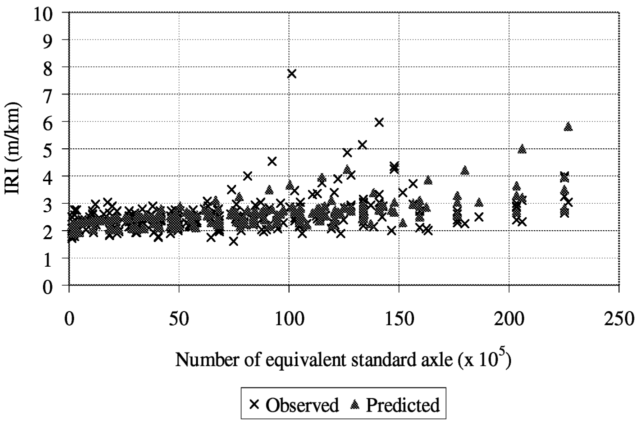
$$IRI = \left[a + b \times REF - c \times \exp(AGE)\right]^{-1} \tag{5}$$

Where
REF = a dummy variable (W_1) having a value of *REF* = -1 if *TOV* < 10cm; or *REF* = 0, otherwise;
D = deflections measured at the loading center (in 0.01mm) after mill and fill, induced by FWD at a load level of 40kN;
AGE = the pavement overlay (mill and fill) age in years;
a, b, c, and *d* = parameters of the model (see Table 3 for values).

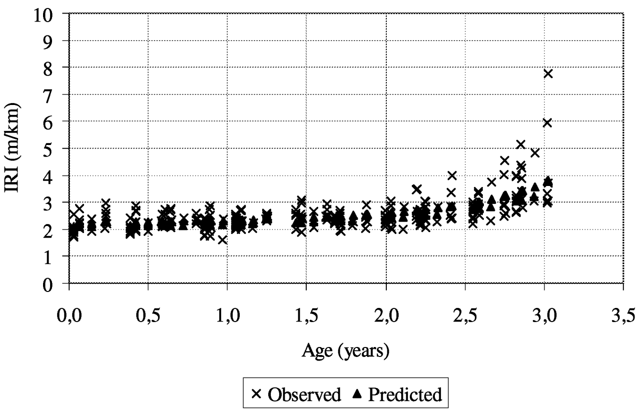
Comparisons between the predicted and measured *IRIs* for the test sections are presented in Fig. 5 and decent fit is seen in the figure. Comparisons were also made on results obtained from Eq. (4) and those predicted by models listed in [1, 2]. A restored section was selected as an example section with the following properties: an overlay thickness less than 10cm; an initial *IRI* value of 2m/km; a deflection of 0.60mm; a structural number of 3.72 and one million ESALs at the first year of analysis. The results show that the *IRI* values predicted by Queiroz´s model for Brazilian road pavements are less conservative. This could be caused by the fact that variables such as deflections, structural number, state of maintenance which is of particular interest in this study by considering the macadam bases erosion, asphalt concrete type, and collinear parameters (age and traffic) were included in the Queiroz´s model. On the other hand, the results obtained from the simplified model in [1] which the deflection was determined by Benkelman's beam are close to the proposed model. It's important

Table 2. Set of Numerical Co-variables Used to Build the Models.

Y_n	X_{ik}							
	$i = 1$	$i = 2$	$i = 3$	$i = 4$	$i = 5$	$i = 6$	$i = 7$	$i = 8$
IRI $dIRI$ ΔIRI $\%CR$ N	IRI_{before} $\ln IRI$	t_{ov}	N $\ln N$ $\log N$ $(\ln N)^2$	$D \times N$ $D \times \ln N$ $(D \times \ln N)^2$ $(D \times \ln N)^3$ N/SNC $\ln N/SNC$ $(\ln N/SNC)^2$	age $exp\ (age)$	$\%CR$	D D^{-1}	SNC $(1+SNC)^{-1}$ $(1+SNC)^{-2}$ $(1+SNC)^{-3}$ $(1+SNC)^{-4}$ $(1+SNC)^{-5}$ SNC^{-1}



(a) By means of Eq. (4)



(b) By means of Eq. (5)

Fig. 5. Comparison between Observed and Predicted *IRI* Values.

to point out that the models listed in [1, 2] were developed for different contour conditions from those presented in this study. Moreover, pavement test sections in this study were milled and filled with identical thickness of new bituminous material, which was not the same as the models developed in [1, 2]. Note that, the devices used to measure roughness in this study were different from those in [1, 2].

Cracking Initiation and Progression (*CR*)

Table 3. Estimations and Significances of the Parameters.

Model	R^2	Parameters	Parameter value Estimation	p-value
(3)	90.4	*a*	- 2.383	0.004
		b	0.102	0.042
		c	0,862	0.000
(4)	31.9	*a*	0.685	0.000
		b	0.076	0.011
		c	- 0.0000000157	0.000
		d	- 0.012	0.020
(5)	54.3	*a*	0.486	0.000
		b	0.031	0.075
		c	- 0.00929	0.000
(6)	64,4	*a*	5.716	0.000
		b	0.189	0.002
		c	0.033	0.029
(7)	54,8	*a*	- 3.988	0.001
		b	3.092	0.000
		c	0.377	0.000

For cracking prediction, two different models were developed as recommended by [1, 2]. The purpose of the first model was to predict the cracking initiation (the occurrence of the first crack) at a specific number of accumulated ESALs (N as the dependent variable). In this case, the overlay thickness (mill and fill thickness) and the modified structural number were treated as the most important independent variables. The prediction equation is defined below:

$$\log_{10} N = a + b * SNC + c * TOV \tag{6}$$

where

N = the number of accumualted ESALs when the fisrst crack occurs

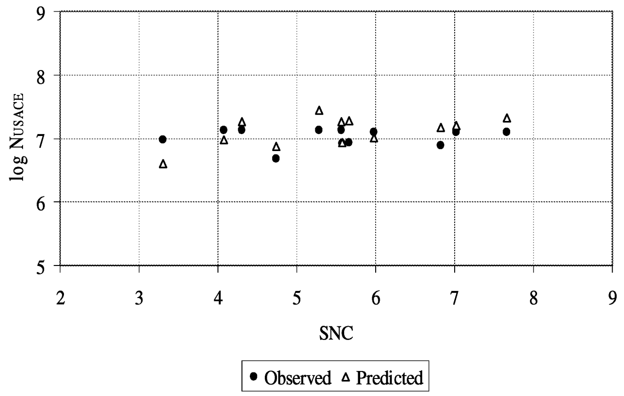
SNC = the modified structural number,

TOV = the mill and overlay thickness (in cm), and

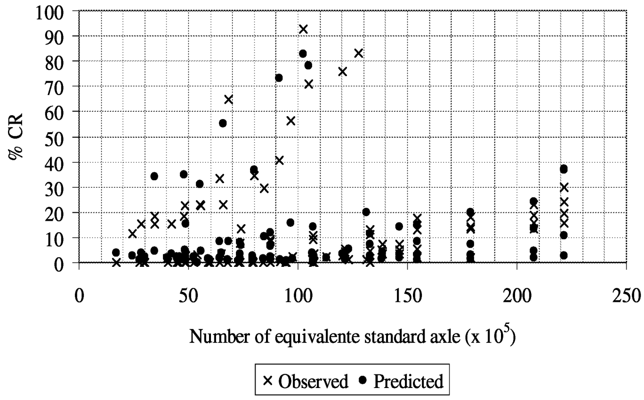
a, b, and c = parameters of the model (see Table 3 for values).

Fig. 6(a) shows the comparison between the observed values and those predicted by the proposed model. The adequate prediction power is shown in the figure.

The second model was developed for cracking progression estimation and the response variable was defined as the percentage of cracked area ($\%CR$) at a fixed number of ESALs.



(a) By the Eq. (6)



(b) By Eq. (7)

Fig. 6. Comparison between the Observed and Predicted Values.

The model is expressed as:

$$\% CR = \exp\left[a + b \times \ln IRI + c \times D \times \ln N\right] \tag{7}$$

Where

$\%CR$ = the percentage of cracked area (%),

D = deflections measured at the loading center (in 0.01mm) after mill and fill, induced by FWD at a load level of 40kN;

N = the number of accumualted ESALs

a, b, and c = parameters of the model (see Table 3 for values).

According to Queiroz [2], this performance model is useful in predicting the cracking condition in a pavement for a period of t years under regular maintenance (the results could be used as a tool to define additional resources required for new projects). Fig. 6(b) presents a comparison between the observed values and those predicted by Eq. (7). $\%CR$ predicted by the proposed cracking model was also compared with the $\%CR$ obtained from the model in [2] for a section with following properties: the same level of traffic; an overlay thickness less than 10cm; roughness of 3m/km; deflection of 0.60mm; and a structural number of 3.72. It was observed that the cracking predicted by model [2] was approximately 27% greater than the $\%CR$ predicted by Eq. (7).

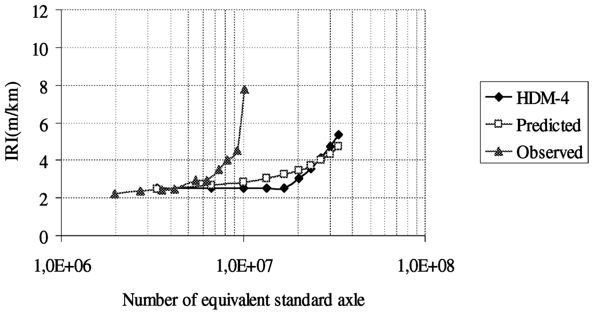
As by monitoring the performance of the test section, it was noticed that initial cracks would take months to be produced. However, the deterioration rate grew relatively fast once the sections started to crack, especially for those sections on the eastbound lanes.

Comparison to the HDM-4 Performance Models

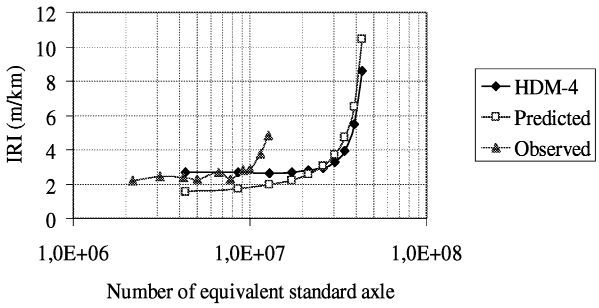
A critical analysis of results was carried out by comparison of the developed models with models suggested by the HDM-4 program [5] for the local climatic conditions (tropical hot-wet), types and load of vehicles and functional and structural characteristics according to the parameters surveyed during the research. HDM-4 works internally with a collection of many performance models developed around the world. The HDM-4 program selects the internal model according to several parameters identified by users, such as the climate regimen (tropical hot and wet), the existing pavement structure (asphalt over granular base), type of overlay (mill and fill), among others. The sections analyzed by the HDM-4 were S1 (right lane), S2 (right lane), S7 (right lane), and S7 (right central lane). An overlay, proceeded by milling, and replaced by equal thickness of HMA strategy was analyzed for a time period of 10 years, and the average daily traffic and types of trucks were calibrated for the analysis on the basis of the typical vehicles observed within the sections. Load equivalency factors were supplied by user and, in this study, the AASHTO criteria was used for calculations ("fourth power law"). For each strategy, HDM-4 was used to simulate deterioration conditions of those sections every year. From Fig. 1, it can be deduced that the predicted *IRI* after mill and fill (IRI_{after}) by Eq. (3) are very to close to those measured in the field. The predicted value of *IRI* by HDM-4 (after mill and fill) is of 3.2 m/km in the right lane of section S7, which was 20% higher than the observed value.

To analyze the roughness progression, comparisons of field measured values with those predicted by Eq. (4) and by the HDM-4 (simulated for a deflection of 0.31mm) are illustrated in Fig. 7. In general, *IRI* values predicted by Eq. (4) are comparable to those estimated by the HDM-4 for sections with an overlay thickness less than 100mm. However, the measured *IRI* values increase rapidly before the values predicted by both the proposed model and HDM-4 models. In section S1, the predicted roughness values (at ten years) by models are lower than the measured value of 7.8m/km. For the section with an overlay thickness greater than 100mm, according to Fig. 7(c), the proposed model was less conservative. The prediction made by the HDM-4 model shows a better fit to the measured data; nevertheless, predicted roughness after mill and fill was larger than the observed value.

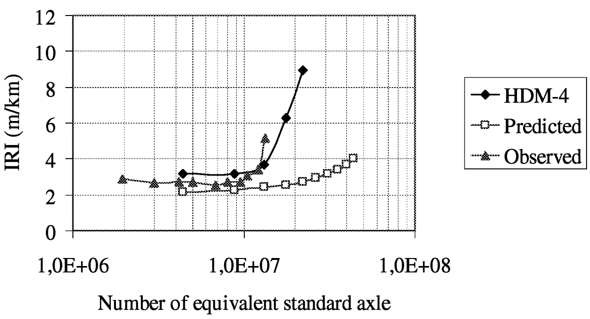
For prediction for cracking initiation (the occurrence of the first crack), values obtained from Eq. (6) were compared to those computed by HDM-4 model. Eq. (6) did produce values that show better fit to measured values, as presented earlier in the paper. For the right central lane in section S7, simulation with the Eq. (6) was carried out for a modified structural number of 5.8. For cracking progression, Fig. 8 presents comparisons of measured values with predicted ones by Eq. (7) and HDM-4 model. HDM-4 model resulted in less conservative values as predicted that cracking would not occur after ten years of services for right center lane of sections S2 and S7. In section S1 the measured values were higher than the predicted ones. The maximum cracking percentages predicted by the models were around 12 to 15% (10 years), whereas the observed percentages were as high as 93% in three



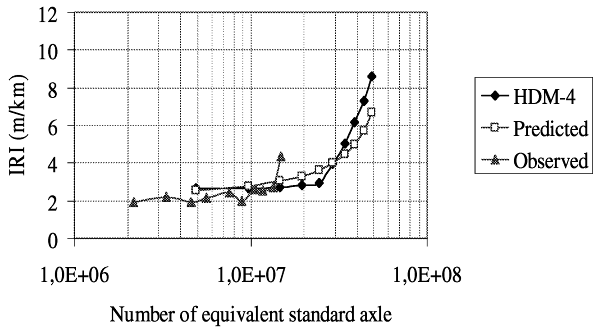
(a) Section S1 (*TOV* = 50 mm)



(b) Section S2 (*TOV* = 80 mm)

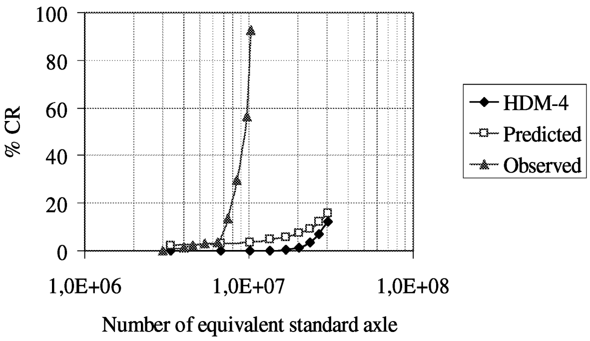


(c) Section S7 (*TOV* = 150 mm)

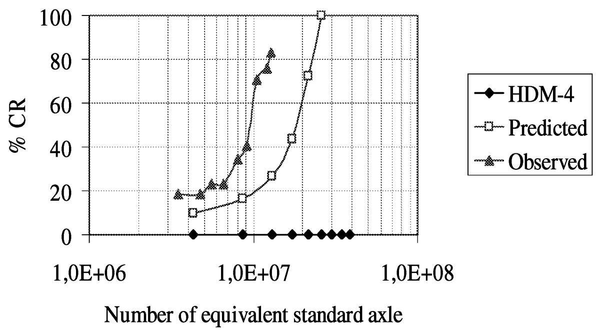


(d) S7 (*TOV* = 50 mm)

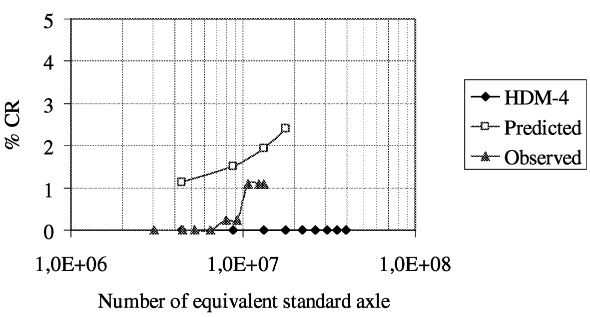
Fig. 7. Comparison between Developed Roughness Prediction Model and HDM-4.



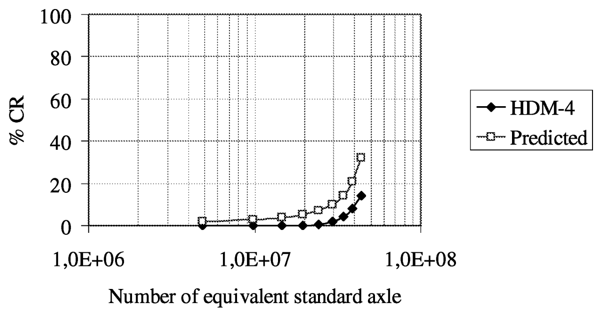
(a) S1 (*TOV* = 50 mm)



(b) S2 (*TOV* = 80 mm)



(c) Section S7 right lane (*TOV* = 150 mm)



(d) S7 right central lane (*TOV* = 50 mm)

Fig. 8. Comparison between Developed Cracking Prediction Model and HDM-4.

years of monitoring. Fig. 8(c) shows that the percentage of the cracked area is less than 3%. Although roughness was taken into account in the cracking prediction, the prediction provided by the proposed model, Eq. (7), was the closest to observed value. Significant discrepancies between measured *%CR* and those predicted by HDM-4 is attributed to the inability of HDM-4 cracking prediction model to describe the local conditions, such as eroded macadam bases and very quick reflective cracking due to bad structural capacity of sections, which were not considered in HDM-4 model.

Conclusions

The primary objective of the study was to understand the causes of the rapid deterioration of the roads in São Paulo City after subjected to rehabilitation. To achieve this purpose, an evaluation of the deterioration process in asphalt overlays in roadway pavements subject to high volume of heavy traffic, as encountered in urban environment, along with a history research and the systematic monitoring of pavements in Bandeirantes were conducted. Performance prediction models were developed for two types of deterioration, roughness and cracking. The developed performance models are intended to be used as pavement system management tools by urban road agencies. Therefore, parameters selected in developing these models were those that can be physically measured and which can be obtained within affordable budget policy. Based on results of the study, the following conclusions can be drawn:

1. In the past, little attention was given to the main reason for poor roadway pavement performance – the contamination by fines in the base layer. Pumping has been evident along the roadways. This was confirmed during the field investigations by opening trenches. The existence of poor subgrade foundation resulted in the poor performance of subsequent rehabilitation activities, such as overlay. The common components for main urban road systems within the city consist of bad drainage and weak subgrade foundation, with resilient modulus no greater than 40MPa.
2. The rehabilitation in 2002 improved the riding quality of most pavement test sections. However, a few sections had poor roughness after mill and fill operations due to lack of construction quality (poor compaction of asphalt mixtures). The reduction in roughness, as measured by *IRI* in the field had good agreement with values predicted by the HDM-4 model.
3. Taking into account *Autoregressive of order #1 type covariance structures* for statistics and the *parsimony* principle, two models for roughness prediction and one model for cracking prediction were developed.
4. The proposed cracking progression model predicts an increase rate of cracks that is slower than that predicted by the model currently adopted by Brazilian Highway Federal Agency ([2, 4]).
5. The proposed model for prediction of roughness immediately after mill and fill compared well to field measured roughness data than the HDM-4 model did. This might be due to the fact that the HDM-4 model was not calibrated with local conditions.
6. Overlay thickness was found to be a factor that influences the comparisons between the proposed models and the HDM-4 models.
7. The proposed model for prediction of cracking initiation matched better to field data than the HDM-4 model did. This might be due to the fact that HDM-4 cracking prediction model did not take into account the condition of much damaged macadam bases in its model. Reflective cracking would develop quickly after repair. Therefore, for preventive maintenance purposes, the developed model should be used by local road agency for their management of maintenance activities.

Recommendations

1. In the face of obvious doubts among roadway agencies in urban environment in Brazil about the applicability of the HDM-4 model under urban road conditions since it was developed for highway pavement analysis, this study indicated that, when roughness became the main technical parameter for economical evaluations, the HDM-4 is an adequate toll to be used under urban road conditions in Sao Paulo.
2. As for cracking progression, the developed models in this study provided more reasonable prediction of future pavement conditions than that provided by the model prescribed in HDM-4. The cracking progression models developed in this study are therefore recommended for implementation in Sao Paulo City PMS as a progressive standard for pavement performance analysis and overlays design. Continued monitoring of future pavements designed in accordance with this new standard will be strongly recommended to permit the collection of more field data that will be used for calibration of the developed models.
3. It is important to emphasize that the models proposed in this study are developed specially for the conditions encountered in the roadway systems in large urban cities in Brazil, such as Sao Paulo, and should be implemented as such.

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