Calibration of HDM-III Performance Models for Use in Pavement Management of South African National Roads

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The applicability of the HDM-III performance models for predicting local observed deterioration was evaluated so that the timing, type, and cost of maintenance needs could be estimated and a balanced expenditure program developed for South African national roads. The validation procedure and the assessment methodologies used in the calibration of the environmental influences and the cracking, rutting, and roughness models are presented. For the HDM-III performance models evaluated, calibration values of less than one were obtained—except for rutting—indicating, in general, better performance on South African national roads than predicted. Based on the results obtained, it is concluded that after calibration the HDM-III performance models are capable of accurately predicting the observed deterioration on South African national roads, and it is recommended that these models be considered for incorporation into a balanced expenditure program for the national road network of South Africa.

The primary road network in South Africa has been established over the last half century and has been planned, constructed, and maintained to provide an acceptable level of service. However, the acute shortage of funds for roads in South Africa is endangering the integrity of this network, putting a considerable emphasis on rationalizing planning in the area of pavement maintenance and rehabilitation. Thus, pavement management, defined as *the total range of activities required to provide the pavement portion of the public works program* (*1*), has become more important.

An essential activity of pavement management is the modeling of the changes in pavement condition with accumulated use, generally known as pavement deterioration. The pavement management system used on national roads in South Africa does not yet incorporate these pavement deterioration prediction models. At present, the current condition of a pavement is used as a trigger for action to identify maintenance or rehabilitation projects for further evaluation. As illustrated in Figure 1, this method has a low probability of selecting the optimum rehabilitation strategy if the expected future deterioration of a pavement is not considered. Although both Pavements A and B in Figure 1 have the same level of riding quality after *T* years, their expected future deterioration differs to a large extent. This demonstrates the need to use deterioration prediction models in pavement management systems to predict the timing, type, and cost of future maintenance needs.

An extensive study (*2*) was executed to evaluate the applicability of models developed internationally for predicting the deterioration of the South African national road network. The study consisted of a literature review of international deterioration models developed from the deterioration results of in-service pavements under the normal traffic spectrum, avoiding models developed from accelerated testing with stationary devices. The reasons for avoiding these models are that the long-term effects are virtually eliminated (they are primarily environmental but also include effects of the rest periods or vehicle headway) and that the unrepresentative traffic loading regimes can distort the behavior of the pavement materials, which is often stress dependent (*3*). From the literature review, the HDM-III models were identified as possibly applicable and were subsequently validated through an analytical approach using the data obtained under the normal traffic and environmental conditions experienced over the past 15 years on the national road network of South Africa.

The aim of this paper is to demonstrate the suitability of the HDM-III performance models, once calibrated, for predicting the performance of South African national roads. After presenting the validation procedure and the assessment methodologies, the calibration of the environmental influences and the cracking, rutting, and roughness models are presented.

VALIDATION PROCEDURE

The approach embarked on during the validation was to first calibrate the environmental coefficient (*m*) for the different Thornthwaite moisture regimes in South Africa, then the HDM-III deterioration models. Based on the fact that the equations defining the different HDM-III deterioration models are of the exponential type, it follows that the accuracy of any prediction tends to decline as the time period increases. Thus, the value of the local calibration factor determined for each model would be valid only over the medium term. Based on this, it was decided to employ the same approach as that used in Chile to adapt the HDM-III models to their local conditions (*4*). The approach involved the periodic calibration of the models to correct the deterioration curves in such a way that they maintained good predictions over the pavement life. Since this was a very time-consuming process, the algorithm developed in Chile that performs the calibration was adopted for South African conditions, to automatically determine the calibration factor for each deterioration model of an individual pavement section.

The calculation method employed in the algorithm is based on the minimization of the difference between the values predicted by the HDM-III models and those measured (*4*). The procedure of cali-

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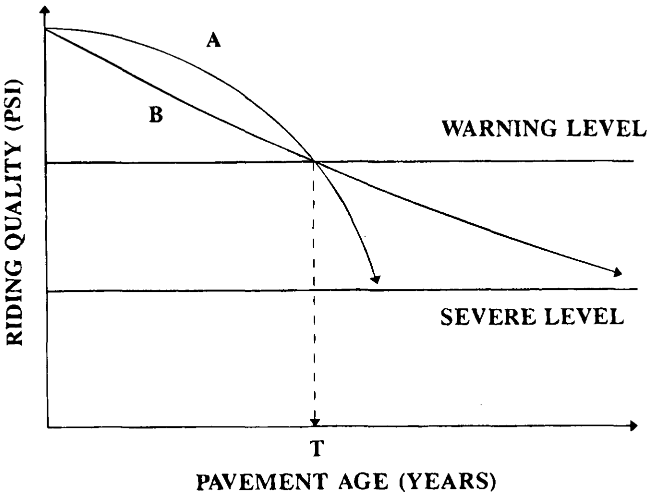

FIGURE 1 **Expected future pavement deterioration.**

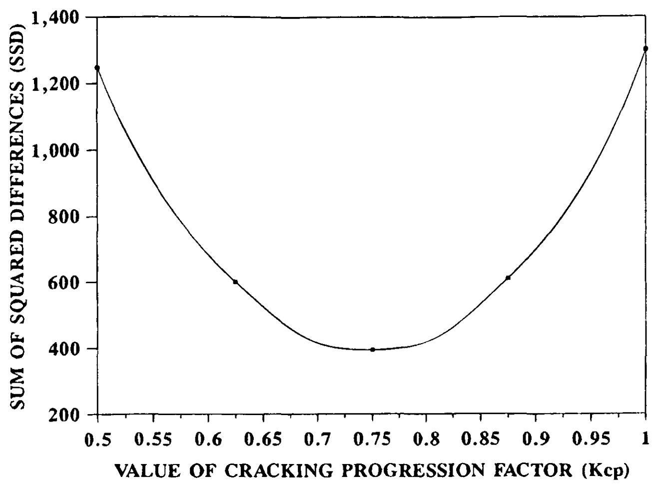

FIGURE 2 **Illustration of sum of squared differences against calibration factor values.**

bration involves the prediction of the change in a specific parameter over time for different calibration factor *ki* values, and then to calculate the corresponding difference between the predicted and measured values for each calibration factor value. These calculated differences are then used to determine the sum of the square of the differences that are then plotted against the specific calibration factor value. When plotted, the sum of squared differences are distributed in a parabolic shape, with a minimum at the optimum calibration factor value, as illustrated in Figure 2 for the cracking progression calibration factor (*kcp*). A parabolic curve is then fitted to the sum of squared differences (*SSD*) incorporating the calibration factor (*ki*) as follows (*4*):

$$SSD = aki^2 + bki + c \tag{1}$$

where

SSD = sum of the squared differences,
ki = calibration factor, and
a,b, and c = constants of equation obtained during the fitting of the curve.

The value obtained by taking the derivative of the equation above equals the calibration factor (*ki*) for which the *SSD* is the least, namely,

$$ki = \frac{-\mathrm{b}}{2a} \tag{2}$$

where

ki = calibration factor for which SSD is a minimum, and
a and b = constants obtained during the fitting of the parabolic curve.

The procedure above was repeated for all prediction models for each individual pavement section evaluated.

CORRELATION OF VISUAL ASSESSMENTS

Since the HDM-III model requires the area affected by cracking (all and wide) and raveling as a percentage value, the correlation of South African visual data (*2*) was needed to convert the degree and extent numerical ratings into percentage of area. Both the extent and degree numerical ratings were combined into a single value, defined as the area of indexed distress. The reason for adopting this approach is that it is believed that by incorporating both degree and extent in a single value, the value obtained will more accurately portray the pavement condition. The following conversion factor was used to convert the South African numerical ratings into the format required by the HDM-III model for each pavement section:

$$\mathrm{CRX} = \frac{\overset{N}{\Sigma}\, A_{TX} S_{TX}}{N} \tag{3}$$

where

CRX = total area of indexed distress as a percentage of the surface area of the pavement section under evaluation;
A_{TX} = area of surface distress for a certain degree as a percentage;
S_{TX} = decimal factor assumed for converting the degree rating; and
N = number of visual segments in the pavement section under evaluation.

CORRELATION OF MECHANICAL MEASUREMENTS

Of the three mechanical measurements of importance on national roads, rutting as well as deflection was already in a format suitable for inclusion in the HDM-III model. The most important of these measurements, namely, the riding quality, had to be converted from present serviceability index (PSI) to quarter-car index (QI_m).

The roughness measurements in PSI, available on the data base, were used to calculate the mean PSI value for each section for a specific survey date. These mean PSI values were then correlated to the quarter-car index (QI_r) by using the correlation developed by Visser (*5*), and then to QI_m by using the relationship in Table 2.5 of Paterson (*3*). (Note QI_m equals 13 on the international roughness index).

$$QI_r = 92.63 - 56.93\ln(\text{PSI}) \quad (4)$$
$$QI_m = 9.5 + 0.9QI_r$$

where

QI_r = quarter-car index [profile RMSVA function of QI (counts/km)];
QI_m = quarter-car index [roadmeter estimate of QI roughness (counts/km)]; and
PSI = mean PSI value calculated for each pavement section.

MODIFIED STRUCTURAL NUMBER

The modified structural number (SNC), which includes the contribution of the subgrade (SN_{sg}), was calculated by using the following equation:

$$\text{SNC} = SN_i + SN_{sg} \quad (5)$$

where

SNC = modified structural number;
SN_i = initial structural number in first year of modeling;
SN_{sg} = contribution of the subgrade after Hodges et al. (*6*): = $3.51 \log_{10} \text{CBR} - 0.85 (\log_{10} \text{CBR})^2 - 1.43$; and
CBR = in situ California bearing ratio of subgrade in percentage.

The initial structural number (SN_i) was determined by using correlations developed by Rohde (*7*), whereby a pavement's structural number can be determined from its total thickness and the shape of the measured surface deflection bowl obtained from a falling weight deflectometer (FWD). The correlations are based on the general "two-thirds rule" suggested by Irwin (*8*) to explain the stress distribution and thus origin of deflections found below an FWD. This rule is based on the fact that approximately 95 percent of the deflection measured on the surface of a pavement originates below a line deviating 34 degrees from the horizontal. Based on this simplification, it can be assumed that the surface deflection measured at an offset of 1.5 times the pavement thickness originates entirely in the subgrade. By comparing this deflection with the peak deflection, the following index associated with the magnitude of deformation that occurs within the pavement structure was defined by Rohde (7):

$$\text{SIP} = D_0 - D_{1.5Hp} \quad (6)$$

where

SIP = structural Index of the pavement;
D_0 = peak deflection measured under a standard 40-kN FWD impulse load;
$D_{1.5Hp}$ = surface deflection measured at an offset of 1.5 times Hp under a standard 40-kN FWD impulse load; and
Hp = total pavement thickness.

To develop a relationship between FWD-measured surface deflection and a pavement's structural number, 7,776 pavement structures were analyzed using layered elastic theory. The SN for each pavement was calculated by using the following approach suggested by AASHTO (*9*):

$$\text{SN} = 0.04\Sigma a_g h_i (E_i/E_g)^{1/3} \quad (7)$$

where

SN = structural number;
a_g = material and layer strength coefficients, per inch;
h_i = layer thickness, mm (where $\Sigma h_i \leq 700$ mm);
E_i = resilient modulus of pavement layer; and
E_g = resilient modulus of standard materials in the AASHTO Road Test.

By comparing the calculated SN with the parameters previously defined, Rohde (*7*) obtained the following relationship between SN and SIP:

$$\text{SN} = k_1 \text{SIP}^{k2}\ \text{Hp}^{k3} \quad (8)$$

where

SN = structural number;
SIP = structural index of the pavement (in μm);
Hp = total pavement thickness (in mm); and
ki = coefficients as listed in Table 1.

The acceptability of SN determined according to the foregoing procedure was continuously verified by using the approach suggested by AASHTO (*9*). Where noticeable differences existed between the two methods (e.g., unrealistic high SN predicted according to procedure above, normally associated with unrealistic low deflections), the SN determined according to the AASHTO (*9*) approach was used.

Finally the initial structural number SN_i was defined as the structural number calculated according to procedure above less the contribution of any maintenance actions within the period between the date of the FWD measurements and the date used as the initial year of modeling:

$$SN_i = \text{SN} - 0.04\ \Sigma\ a_i h_i \quad (9)$$

where

SN_i = initial structural number in first year of modeling;
SN = structural number;
a_i = material and layer strength coefficients, per inch; and
h_i = thickness of overlay, reseal, and so forth, in mm.

ENVIRONMENTAL ROUGHNESS CALIBRATION FACTOR (*Kge*)

The environmental roughness calibration factor (*Kge*) is the exponential annual rate of increase in roughness due to environmental effects. The environmental roughness calibration factor (*Kge*) is calculated from the environmental coefficient (m) as follows:

$$Kge = m/0.023 \quad (10)$$

where

Kge = environmental roughness calibration factor; and
m = environmental coefficient.

Advice as to recommended values for the environmental coefficient (m) for various climatic regions is given in Table 8.7 of Paterson (*3*). However, Paterson (*3*) cautions that these recommended

TABLE 1 Coefficients for SN Versus SIP Relationships (7)

Surfacing type	k1	k2	k3
Surface Seals	0,1165	-0,3248	0,8241
Asphalt concrete	0,4728	-0,4810	0,7581

values are based on relatively few evaluations. Based on this and the advice given by Paterson during the Botswana calibration of HDM-III, it was decided to follow his recommended method for determining a value for the environmental coefficient (m) from roughness measurements.

The recommended procedure first runs HDM-III with the environmental roughness calibration factor set at zero. This establishes the contribution of traffic to the increase in roughness. The environmental coefficient (m) is then approximated by dividing the difference between the total increase and the increase due to traffic by the product of the mean roughness and the number of years since construction. In mathematical terms:

$$m = \frac{(Rm - Ri) - (Rp - Ri)}{(Rm + Ri) \times (T/2)} \tag{11}$$

where

Rm = measured roughness;
Ri = initial roughness;
Rp = predicted roughness with $Kge = 0 = m$; and
T = number of years between measurement date and construction date.

For this study, the pavement sections under evaluation were subdivided into the different Thornthwaite moisture regimes occurring on South African national roads. Since multiple observations existed under each moisture regime, the best estimate for m was given by the quotient of the sums of the individual numerators and denominators. The results obtained for the different moisture regimes are summarized in Table 2.

As seen, the calculated environmental roughness calibration factor (Kge) in each instance is nearly half of the value recommended by Paterson (*3*) for that moisture regime. Thus, the influence of the environment on the pavement deterioration observed on South African national roads, is only about half of what is predicted by the HDM-III model. Possible contributing factors include the following.

- The generally more balanced deep pavement structures used in South Africa, which result in more support for the surface layer, and, as such, decrease the induced stresses within the upper layers. This results in a longer period before initiation of cracking. This is in contrast to the relatively shallow pavements used during the development of the models.
- The design and quality control during construction in South Africa, which result in a high-quality finish with adequate provision for surface as well as subsurface drainage.
- The maintenance activity employed on South African national roads, which include routine activities such as crack sealing and periodic overlays or reseals, which decrease the environmental influences, thus increasing the life of a pavement.

CRACKING MODEL

Cracking is modeled in two phases: the time before initiation of cracking and the rate of progression of cracking for both all and wide cracking. The cracking model relates the change in cracking to

Incremental cracking area = Kcp {Kci f(equivalent standard axles, construction quality, structural strength, base type) + f(area previous cracking)}

where

Kci = user-defined factor for local calibration of all cracking initiation; and
Kcp = user-defined factor for local calibration of all cracking progression.

The reason for not having calibration factors for wide cracking, is that the initiation of wide cracking was defined as a function of the initiation of all cracking.

Initially the pavement sections were evaluated individually based on the surfacing type, base course type, and climatic area. No noticeable difference in performance between the different surfacing layers or base course layers existed. It is believed that for the surfacing, this is the result of using asphalt layers on South African national roads that are normally thin (30–40 mm) and using Cape seals as surface treatments. The Cape seal has one or more slurry

TABLE 2 Environmental Coefficient (m) for Different Moisture Regimes

Moisture regime	Semi-Arid	Subhumid	Humid
Calculated value for m	0,009	0,014	0,020
Calculated value for Kge	0,392	0,607	0,886
No of observations	20	25	20
Paterson (*3*) value for kge	0,70	1,30	1,74

seals on top of the 19-mm single seal aggregate. This improves the impermeability of the layer and also limits raveling to a large extent, which leads to an improvement in the performance of the seal. For these reasons, similar performance of the surfacing types were found. For the different climatic areas, no noticeable difference existed in the cracking initiation and progression calibration factor values. Thus, the calibration values for cracking initiation and progression were evaluated for all pavement sections regardless of surfacing type, base course type, or climatic area. The only noticeable difference was between original constructed pavements, and those with overlays or reseals.

The cracking initiation calibration values (*Kci*) for original constructed pavements followed a normal distribution with an average value of 1.41 and a standard deviation of 0.59 on 65 roads. This indicates that the period until the initiation of cracking on South African national roads is longer than the period predicted by the HDM-III model for the same volume of traffic. For overlays or reseals, the cracking initiation calibration values also followed a normal distribution, with an average value of 0.63, and a standard deviation of 0.17 (31 roads). This indicates that the expected life until cracking initiation tends to be lower for an overlay than the value predicted by the HDM-III model for the same traffic volume. It is believed that this is the result of the average overlay thickness of 30–40 mm generally used in South Africa being less than the average overlay thickness of 50–125 mm used in the Brazil study, from which the HDM-III cracking models were developed. This thinner layer thickness results in a shorter propagation length for cracks, with a subsequent faster rate of cracking initiation for South African overlays.

The cracking progression calibration values (*Kcp*) for original surfacings also followed a normal distribution, with an average value of 0.21 ($\sigma = 0.08$) indicating that the progression of cracking observed on national roads is lower than the rate of progression predicted by the HDM-III model for the same volume of traffic. The same was applicable for overlays and reseals, with an average value of 0.59 ($\sigma = 0.40$). Possible factors contributing to the aforementioned observations are as follows:

- The routine maintenance program employed ensures that a road is sealed or overlaid within an average of 8 years. This activity severely limits the probability for cracking initiation and progression to the severe rates observed during the Brazil study, as is evident in the low areas of cracking observed.
- In South Africa, the asphalt type used is of a semigap grading, whereas the type generally used in the Brazil study was continuously graded. It is known that a semigap graded asphalt is more resistant to fatigue than the continuously graded, resulting in a longer period before the initiation of cracking and a slower rate of progression once initiated.

The HDM-III model predictions after calibration compare favorably with the observed values, as is evident from Figure 3 with an R-squared value of 0.91 obtained for original surfacings and Figure 4 with an R-squared of 0.94 obtained for overlays and reseals. Since only a limited number of sections with relatively larger areas of cracking were available, it was impossible to evaluate the prediction of area by the HDM-III models for larger areas.

Thus it was concluded that for the low areas of cracking observed on national roads, the HDM-III model predictions after calibration seem reasonable. It is recommended that the ranges in Table 3 be used in the selection of a calibration factor value, if an individual value is not available for the specific section.

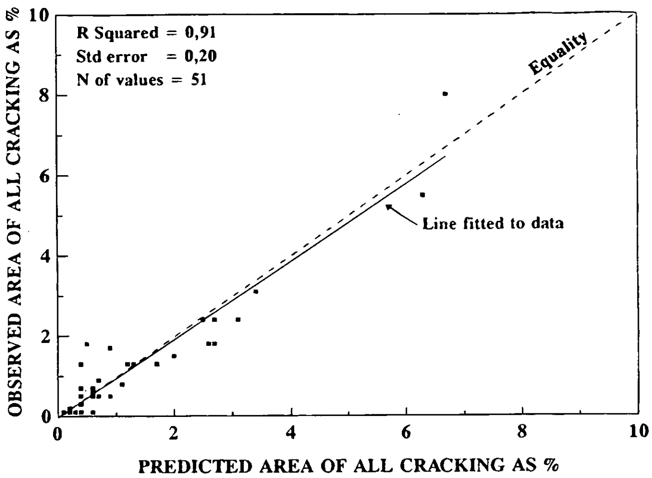

FIGURE 3 Comparison between predicted and observed values for area of all cracking for original surfacings.

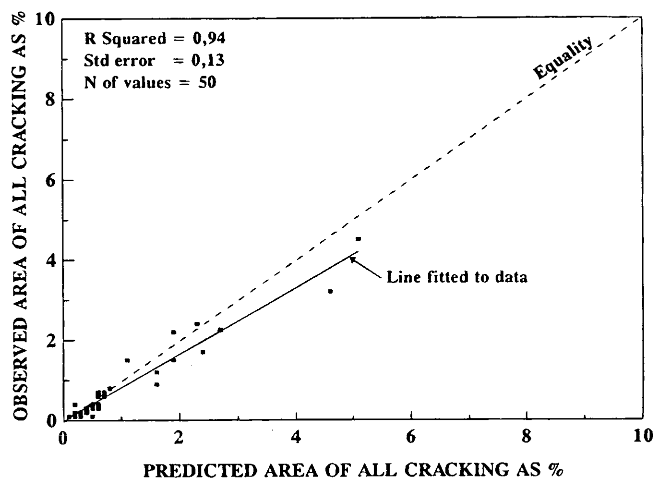

FIGURE 4 Comparison between predicted and observed values for area of all cracking for overlays and reseals.

RAVELING MODEL

Raveling is also modeled in two phases, namely, the time before initiation of raveling, and the rate of progression of raveling. The raveling model relates the change in raveling to

Incremental raveling area = *K*vi *f*(surfacing type, construction quality, traffic) + *f*(previous area of raveling)

where *Kvi* is the user-defined factor for local calibration of raveling initiation.

Since no accurate method existed for correlating the historic data, no correlation values could be determined for the raveling model of the HDM-III model. It is believed that this would not affect the calibration values of the other models adversely, since the influence of raveling on potholing is of importance only when the area of raveling exceeds 30 percent, which never occurred on the sections eval-

TABLE 3 Recommended Range for Calibration Factor Values of the Cracking Model

Pavement type	Cracking initiation (Kci)	Cracking progression (Kcp)
Original surfacings	1,00-1,50	0,1-0,3
Overlays and reseals	0,4-0,8	0,3-0,7

uated. Thus, it is recommended that for raveling initiation (*Kvi*), a default value of one should be used, until calibration values are determined from a more accurate source of information.

POTHOLING MODEL

The pothole model relates the change in pothole area to

Incremental pothole area = *Kpp f*(wide cracking, raveling, previous pothole area)

where *Kpp* is the user-defined factor for local calibration of pothole progression.

In the HDM-III model, the minimum requirements for the initiation of the pothole models were defined as a minimum area of wide cracking of 20 percent for asphalt surfacings, or a minimum raveled area of 30 percent for surface treatments. As a result of the maintenance activity of patching of all potholes, and the area of wide cracking never exceeding 20 percent for asphalt surfacings or the area of raveling never exceeding 30 percent for surface treatments, the pothole models were never initiated within the HDM-III model. Thus, no method for determining calibration factor values for the pothole models existed, since no predictions were made by the HDM-III model. Thus, it is recommended that for the pothole progression calibration factor (*Kpp*), a default value of one be used until further information becomes available.

RUTTING MODEL

The rutting model consists of the mean rut depth model and the rut depth standard deviation model. The rutting model relates the change in mean rut depth and rut depth standard deviation as follows:

Incremental mean rut depth = *Krp f*(time, equivalent axle load, structural number, compaction, deflection, precipitation)

Incremental standard deviation = *Krp f*(mean rut depth, structural number, compaction, equivalent standard axles)

where *Krp* is the user-defined factor for local calibration of rut depth progression.

The mean rut depth model is not used directly in the HDM-III but is used instead as a means to estimate the variation of rut depth (standard deviation) that contributes directly to the roughness model. For the rutting model, HDM-III allowed for only a user-defined calibration factor for the progression of rutting, *Krp*. Since the use of rut depth measurements at a network level on national roads ceased in 1987, only a limited number of rut depth measurements were available for evaluation.

The calibration values for rut depth progression appear to follow a normal distribution with an average value of 1.57. It is believed that this average value does not necessarily indicate a faster rate of rut depth progression for South African pavements. The reason for this being higher than one is that in South Africa a 2-m straight edge is used compared with a 1.2-m straight edge used in the development of the model. No direct correlation between ruts measured with the different straight edges was found.

In Figure 5, the comparison between predicted values and observed values is illustrated for the mean rut depth, and for rut depth standard deviation in Figure 6. From Figure 5, it is evident that for the limited number of rut depth measurements available on national roads, the predictions given by the HDM-III model after calibration is not that favorable, with an *R*-squared of 0.68 being obtained. From Figure 6, it is evident that the correlation obtained for rut depth standard deviation is even worse, with an *R*-squared value of 0.28 being obtained. The limited data available, as well as the difference in straight edge length, are believed to contribute to the poor correlations. It is recommended that the calibration range in Table 4 be used for the rut depth progression factor (*Krp*).

ROUGHNESS MODEL

This model combines the predictions of all the previously mentioned models into a single value, which forms the basis for deter-

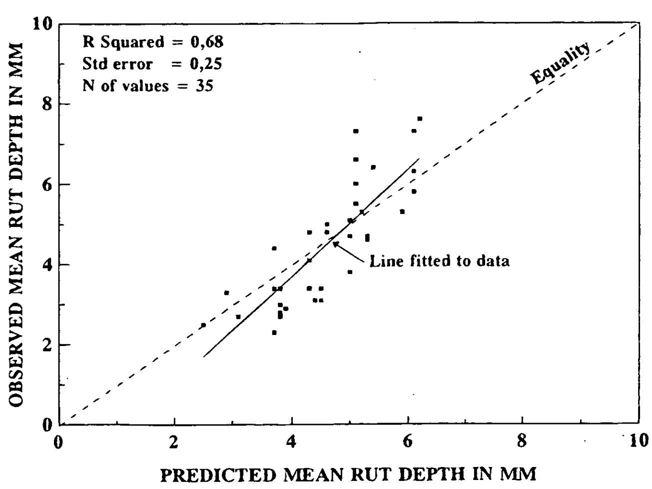

FIGURE 5 Comparison between predicted and observed values for mean rut depth.

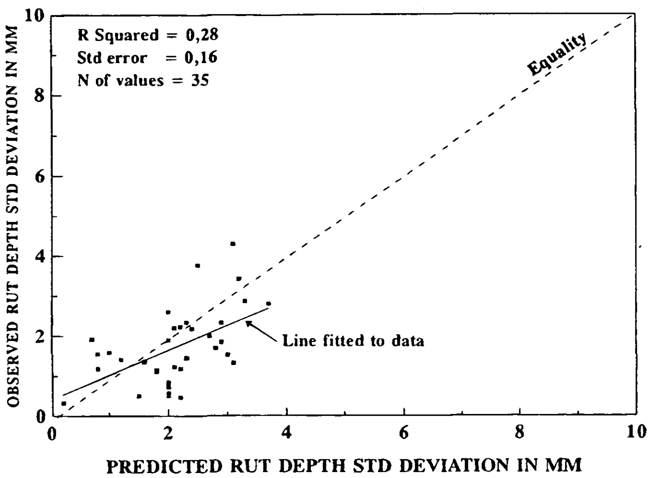

FIGURE 6 Comparison between predicted and observed values for rut depth standard deviation.

mining vehicle operating costs, and economic intervention levels. The incremental roughness model relates the changes in roughness to

Incremental roughness = *Kgp*{*f*(structural number, incremental traffic loadings, extent of cracking, thickness of cracked layer, incremental variation in rut depth) + *f*(changes in cracking, patching, and potholing)} + *Kge*{*f*(pavement environment, time, and roughness)]

where

Kgp = user-specified factor for local calibration of roughness progression, and
Kge = user-specified factor for local calibration of the environment-related annual fractional increase in roughness.

The environment-related calibration factor, *Kge*, is fixed to certain values, defined on the basis of the Thornthwaite moisture index, as discussed previously. Initially, the pavement sections were evaluated individually based on the surfacing type and base course type and whether it was an original constructed surface layer or an overlay or reseal. No noticeable difference in performance existed between the different surfacing layers and base course layers or between original surfacings or overlays and reseals. Thus, only differences in moisture regime were allowed for during the calibration for roughness progression (*Kgp*).

For semiarid areas, the roughness progression calibration factor (*Kgp*) followed a normal distribution with 85 percent of the calibration factor values falling within the range 0.8 to 1.2, with an average of 1.02. The same applied to subhumid areas with 88 percent of the calibration factor values falling within the range 0.6 to 1.4, with an average of 0.95. The aforementioned also applied to humid areas with 70 percent of the calibration factor values falling within the range 0.8 to 1.2, with an average of 0.99. As with the previous two moisture areas, the average value obtained indicates that the observed roughness deterioration on South African national roads is equal to the value predicted by the HDM-III model. Thus, after calibrating the HDM-III model for local environmental conditions, it seems that little or no calibration is needed for the roughness progression model, indicating that the deterioration predicted by the HDM-III for traffic-related distress seems to be similar to the deterioration observed on South African national roads. Furthermore, this indicates that the expected difference in behavior between the different climatic areas is taken into consideration by the environmental roughness calibration factor (*Kge*), which increases or decreases the rate of deterioration as required.

The ability of the HDM-III model to predict the roughness observed on national roads after calibration is illustrated in Figure 7 for all pavement sections evaluated. As seen from the figure, an *R*-squared value of 0.9 was obtained, indicating that after calibration, the HDM-III model is capable of accurately predicting the roughness deterioration observed on South African national roads. Thus, the use of the HDM-III deterioration models for predicting the deterioration observed on South African national roads is highly recommended, as is evident in Figures 8 and 9, in which the observed roughness is compared with the predicted roughness for an individual pavement section evaluated. It is also obvious from these figures that the maintenance activity employed on South African national roads did not allow the evaluation of the exponential nature of the HDM-III models. The reason is that when maintenance is timely, the deterioration of a pavement is kept to more or less a linear progression as seen in Figures 8 and 9.

CONCLUSIONS AND RECOMMENDATIONS

The main conclusion from the comparison of observed values with predicted values, is that the HDM-III models are capable of accurately predicting the observed deterioration on South African national roads, but that for most models calibration is needed for local conditions, especially for the environmental roughness calibration factor (*Kge*).

Despite the favorable correlations obtained for some of the HDM-III models, others could not be calibrated as a result of the lack of suitable South African deterioration data. Thus, for the raveling, potholing, and to certain extent cracking models, additional research should be conducted for determining calibration values for some models, or more accurate calibration values for other

TABLE 4 Recommended Range for Calibration Factor Values of Rut Depth Model

Pavement type	Rut depth progression (Krp)
Original surfacings	1,5-1,75
Overlays and reseals	1,0

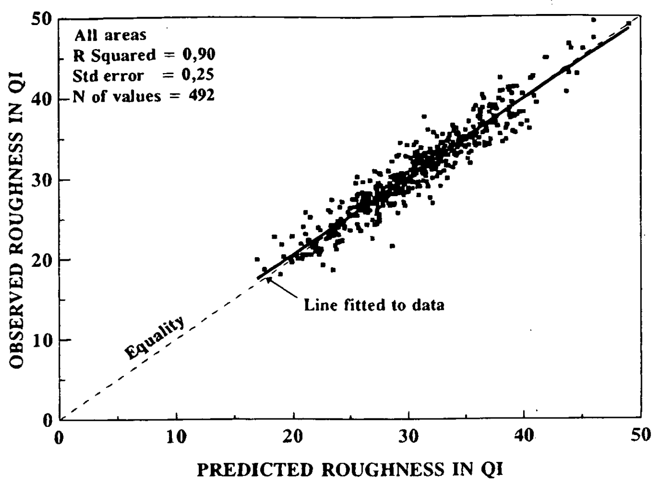

FIGURE 7 Comparison between predicted and observed roughness values for all areas.

models.

Based on the results obtained for the limited number of sections included in the study, it is recommended that the HDM-III models should be considered for incorporation into a balanced expenditure program for the national roads of South Africa. The incorporation of these models would be simple since most of the models only need calibration for them to be applicable to local conditions. The incorporation of these models would allow the prediction of the rate of deterioration of a pavement and the nature of the changes so that the timing, type, and cost of maintenance needs could be

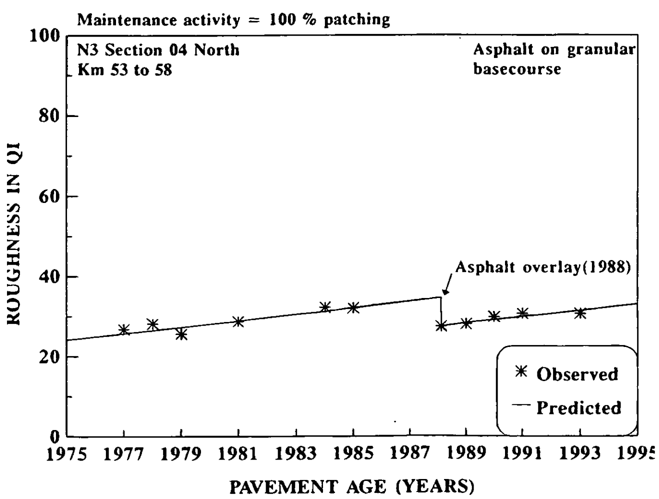

FIGURE 8 Typical illustration of comparison between observed and predicted roughness values for National Route 3, Section 04 North.

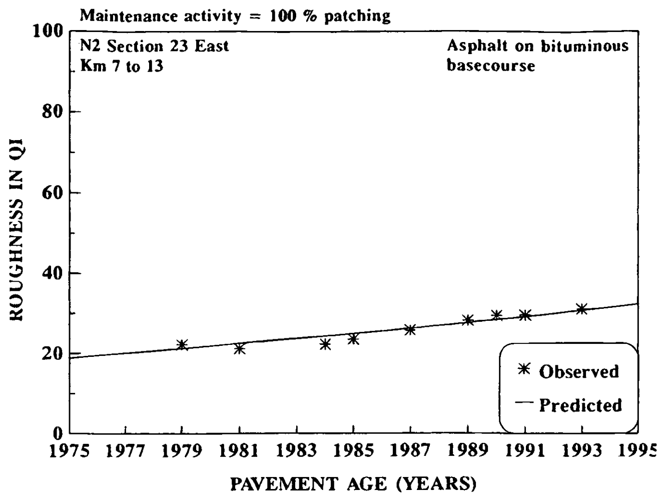

FIGURE 9 Typical illustration of comparison between observed and predicted roughness values for National Route 2, Section 23 East.

estimated.

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