MODELLING THE DETERIORATION OF BITUMINOUS PAVEMENTS IN INDONESIA WITHIN A HDM-4 FRAMEWORK

G Morosiuk
International Division, Transport Research Laboratory, U.K.

T Toole
International Division, Transport Research Laboratory, U.K.

S Mahmud
Institute of Road Engineering, Indonesia

T Dachlan
Institute of Road Engineering, Indonesia

Dr Greg Morosiuk has over 25 years overseas experience with TRL working on a variety of problems concerned with road performance and deterioration, whole life costing and road investment generally. Experience has been gained throughout the world, from the Caribbean, throughout Africa to the Far East. Two years were spent in the Caribbean conducting vehicle operating cost studies. Four years were spent in Malaysia where he was responsible for establishing a Research Institute within the Public Works Department. As the senior road investment and modelling specialist at TRL, he is currently involved in the development of HDM-4.

Tyrone Toole is currently Head of TRL's International Infrastructure and Transportation Department. His direct experience is in highway engineering, road management and institutional development, and the development and direction of research programmes to meet the needs of developing and emerging countries in all aspects of the planning, design, construction, maintenance and safe operation of highways. He has worked throughout Africa, Asia and the Caribbean over a period of 20 years and spent five year based at the Institute of Road Engineering in Indonesia, and three years in Botswana.

Salim Mahmud is Head of the Pavement Research Group at the Institute of Road Engineering and currently leads projects in pavement engineering. His studies have included the calibration of road deterioration models, analytical design of pavements and development of road specifications. Salim is an Engineer who graduated from the Institute of Technology in Bandung, majoring in Transportation and Highway Engineering.

Tatang Dachlan is currently responsible for the evaluation of research and development projects at the Institute of Road Engineering. His experience is in the development and production of standards and guidelines for roads and road building materials, including asphalt mix design. He contributed to the development of improved mix designs employing the Superpave approach and the use of refusal density methods. He supervised the laboratory and field studies described in the current paper. Tatang holds a MEngSc from the University of New South Wales, where he majored in Transportation and Highway Engineering.

1. INTRODUCTION

A comprehensive research programme was undertaken to evaluate the performance of road strengthening overlays used in Indonesia. This work was carried out under a cooperative research programme between the Transport Research Laboratory (TRL) in the UK and the Institute of Road Engineering (IRE) in Indonesia, with funds provided by the UK's Department for International Development (DFID), the World Bank and the Government of Indonesia. Progress has been reported in an earlier paper to this conference (Dardak, et al, 1992).

The objectives of the studies were to enable improvements in mix design and specifications, construction practice and structural design, and to produce road deterioration relationships relevant to the Indonesian environment. The accurate prediction of the rates of deterioration of roads is important in road management systems, to ensure efficient prioritisation and for setting budget levels, and in justifying changes in standards and specifications.

In Indonesia the Integrated Road Management System (IRMS) is currently being improved by the Directorate General of Highways (DGH) for use on the paved road network of the country. This paper describes the performance of sections of road in Indonesia over a ten-year period and compares their rates of deterioration to those predicted by relationships used in the HDM-4 model. The cracking, rutting and roughness relationships are calibrated for the local conditions, thus enabling the IRMS to make use of appropriate deterioration relationships. The benefits arising from improved calibration of models and implications for improvements in asphalt design and specifications are identified. Further details are available in earlier papers (Morosiuk, et al, 1999 and Dachlan, et al, 1997).

2. DESCRIPTION OF THE STUDIES

By the late 1980's the bituminous surfacings on most of the national and provincial network in West Java comprised hot rolled sheet (HRS), produced to the 1986 "Specification for High Durability Asphalt" (DGH, 1986) or asphalt concrete (AC) mixes produced according to Marshall design principles (Asphalt Institute, 1983). The basis for the adoption of HRS was described by Corne (1983) and drew from experience in the UK and elsewhere (BSI, 1985 and 1992 and NITRR, 1978) in the use of gap and semi gap graded mixes. The materials used for HRS were designed to accommodate a larger amount of bitumen than AC-type mixes, the aim being to increase flexibility and durability whilst retaining sufficient deformation resistance.

Although many thousands of kilometres of road have been overlaid with HRS in Indonesia, no quantitative comparison of performance had been available. Such a comparison is of paramount importance to the DGH so that rational decisions can be made on future road maintenance and betterment schemes.

2.1 Evaluation Methodology

The scale of the betterment programme meant that a wide variety of roads had been overlaid under various conditions, enabling a wide range of the key variables to be covered. A "window-monitoring" methodology was used, whereby observations were taken over the monitoring period on sections of road of varying age.

Initially a desk study of design and construction procedures was carried out, followed by general condition surveys of a sample of road links to identify general performance trends and mechanisms of failure. From the results of these general surveys, short sections of road were selected for detailed investigation and time series observations and sampling. In addition supplementary data, such as traffic, climate, maintenance, etc, was collected.

2.2 Experimental Design

Sections of two-lane road, located on five routes, were selected for monitoring, each lane being treated as a separate site. The lanes were divided into 10-metre lengths of road, referred to as blocks, each block being treated as a discrete unit length of road for monitoring and analytical purposes. Blocks on a site that were not typical of the length of road under investigation were omitted from the study, enabling representative and homogeneous lengths to be monitored and analysed.

The condition of the asphaltic mix on each section was assessed through laboratory testing of slabs and cores taken from the sites. This enabled sites to be classified for the purposes of analysis as plastic, normal or brittle according to the voids in the mix (VIM). If VIM was < 3%, the site was classified as plastic; if VIM was > 6%, as brittle; otherwise as normal.

The range of traffic levels, pavement strength and surfacing thickness of these sites are given in Table 1.

Table 1: Experimental Design of the Test Sections

Route	No. of Sites	Pavement Strength (SNC)		Annual Traffic (YE4) (million esa)		Total Surfacing Thickness (HS) (mm)	
		min	max	min	max	min	max
Cirebon – Kuningan	8	4.0	5.1	0.08	0.22	56	126
Cirebon - Losari	8	4.0	7.3	0.92	0.92	90	212
Ciawi - Cianjur	14	2.6	5.9	0.22	0.38	130	240
Tangerang - Merak	14	3.2	8.1	0.65	1.5	66	168
Kopo – Rancanbali	4	2.8	3.5	0.10	0.15	91	131

The strengths of the sections are given in terms of modified structural number (SNC) estimated from the Benkelman beam deflections measurements from the sites using the relationship between SNC and deflections given in HDM-III (Paterson, 1987). The traffic levels are given in terms of the annual equivalent standard axles (YE4), averaged over the duration of the study.

2.3 Monitoring Procedures

A visual assessment of each site was carried out at regular intervals over a ten-year period. The distresses recorded included rut depth (in both wheelpaths using a 2 m straight-edge), area of cracking, potholes and patching, and any other defects that were visible. Other field measurements taken on the sites included roughness (NAASRA meter and DIPSTICK profilometer), pavement deflections (Benkelman beam, FWD) and pavement strength (DCP).

Traffic volumes and axle loadings were derived from traffic surveys conducted by Provincial DGH organisations and by the Directorate of Planning (BIPRAN). Supplementary surveys were undertaken on a number of occasions by the TRL/IRE team to update this information.

3. HDM-4 ROAD DETERIORATION RELATIONSHIPS

The distresses examined in this study were structural cracking, rutting and roughness. The HDM-4 deterioration relationships (Morosiuk, et al, 2000) for these modes of distress are described below.

3.1 Structural Cracking

Cracking in HDM-4 is modelled in two discrete phases (Paterson, 1987). In the initiation phase the distress has not yet become manifest and the area is zero. After initiation the area of cracking gradually progresses following a sigmoidal curve.

Initiation is defined in HDM as the time at which cracking first extends over 0.5 per cent of the area of the road. The cracking initiation relationships predict the time, in years, to 0.5 per cent cracking of the surface area of the road. The cracking progression relationship is a time-based model, predicting the percentage surface area that is cracked, once cracking has initiated.

In HDM-4 separate relationships are given for crack initiation and progression for two classes; 'all' and 'wide' cracking. All cracking includes any cracking that is at least 1 mm wide; wide cracking is defined as cracks that are greater than 3 mm wide or spalled. Separate relationships are assigned to different surfacing and base types.

3.2 Rut Depth

The HDM-4 rut depth model is based on four components of rutting, the rut depth at any time being the sum of the four components. These four components are as follows:

- Initial densification (rutting in the first year after new construction or reconstruction that includes a new base layer)
- Structural deformation (structural rutting in the following years)
- Plastic deformation (shoving in the asphalt layers)
- Wear from studded tyres (rutting from studded tyres used on snow covered roads – applicable only to cold climates)

3.3 Roughness

Roughness progression is predicted in HDM-4 as the sum of five components; structural, cracking, rutting, potholing and environmental components. Roughness is calculated at the end of each year, taking into account the change in condition for each mode of distress sequentially for each year of an analysis period. In this paper a simpler alternative to the HDM-4 incremental recursive approach was used, using the relationship determined by Paterson and Attoh-Okine (1992) which predicts absolute roughness at a point in time, based on the HDM model. This relationship, referred to as the detailed model in this paper, is given below.

$$RI_t = 0.98e^{mt}\,[RI_o + 135SNCK^{-5}\,NE_t] + 0.143RDS_t + 0.0068ACX_t + 0.056PAT_t \quad \ldots (1)$$

where

RI_t = roughness at pavement age t (m/km IRI)
RI_o = initial roughness (m/km IRI)
t = pavement age since rehabilitation or construction (years)

m	=	environmental coefficient
NE_t	=	cumulative esa at age t (millions esa/lane)
SNCK	=	$1 + SNC - 0.00004(HS)(ACX_t)$ for $(HS)(ACX_t) < 10{,}000$
SNC	=	modified structural number of the pavement
ACX_t	=	area of indexed cracking at time t (%) where $ACX = 0.62\ ACA + 0.39\ ACW$
RDS_t	=	standard deviation of rut depths at time t (mm)
PAT_t	=	area of patching at time t (%)

When a general model is required without knowledge of the surface distress, Paterson and Attoh-Okine (1992) developed an alternative relationship for predicting roughness, which is referred to as the aggregate model in this paper. The aggregate model is given below.

$$RI_t = 1.04e^{mt}\ [RI_o + 263(1 + SNC)^{-5}\ NE_t] \qquad \text{. . . (2)}$$

4. CALIBRATION of the HDM-4 DETERIORATION RELATIONSHIPS

The primary objective of this study was to derive road deterioration relationships that are appropriate for Indonesian conditions and therefore could be adapted for use in the IRMS. Hence the analysis examined the universally accepted deterioration relationships in HDM-4, with a view to calibrating them for the local conditions. All the deterioration relationships in HDM-4 have individual calibration factors, the default value of each factor being set to unity.

Furthermore, whilst the initial focus assumed that significant differences in performance related to mix specification would be identified, subsequent investigations (Dardak, et al, 1992) showed that the performance of the overlays was highly correlated with the occurrence of mixes whose properties met normal mix design criteria for deformation resistant and durable mixes. Thus, the analysis described in this paper has focused on individual mix parameters, including air voids, softening point, etc, and traffic loading conditions including vehicle speed and cumulative standard axles.

4.1 Structural Cracking

Relatively little cracking was observed on the sites, with most of the observed cracking being classified as narrow cracks. Therefore only the progression of ‘all’ cracking could be examined. The observed rates of cracking progression were compared with the rates of progression predicted by the HDM-4 model.

The HDM-4 model has two calibration factors for ‘all’ cracking; one is for adjusting the time to initiation of cracking (K_{cia}) and the other is for adjusting the rate of crack progression (K_{cpa}). The HDM-4 predicted rates were adjusted to those observed on the test sites by adjusting the values of K_{cia} and K_{cpa}.

By the end of the monitoring period, sites on three of the five routes had exhibited some cracking, enabling calibration factors for crack initiation to be derived. However only the sites on the Cirebon – Kuningan route had exhibited sufficient cracking to enable calibration factors for crack progression to be derived. The range and average values of these calibration factors are given in Table 2.

Table 2: Cracking Calibration Factors

Route	Crack Initiation - Kcia	Crack Progression - Kcpa

	avg	min	max	avg	min	max
Cirebon – Kuningan	0.5	0.3	0.7	1.1	0.6	1.6
Cirebon – Losari	0.8	0.8	0.8			
Tangerang – Merak	0.7	0.5	0.9			

The values of K_{cia} in Table 2 are relatively consistent for each route, the average values for the three routes ranging between 0.5 and 0.8. These values of K_{cia}, being less than unity, indicate that cracking was observed to initiate earlier than predicted by HDM-4. For the pavement strengths and traffic loadings encountered on the test sites, HDM-4 predicts that cracking will initiate after 6 to 7 years. The values of K_{cia} derived for these sites, indicate that cracking was observed to initiate after 3 to 5 years. Early cracking was shown to occur where in situ air voids remained above 6%. In such cases the penetrations of recovered binders were low (< 30) and the softening points were high (> 60 °C).

Site investigations during this study revealed that bitumen seals were regularly used on cracked areas. Therefore on most sites that had cracked, less than 15% area of cracking was observed. This made the calibration of cracking progression more difficult to deal with in a comprehensive manner. However, the values of K_{cpa} derived during this study, tend to indicate that the rate of crack progression predicted by HDM-4 is in general agreement with the rates observed on the 'brittle' sites, for this limited range of cracking.

4.2 Rut Depth

<u>Structural Deformation</u>

Rutting was examined only on the sections where the geometry was relatively flat. This was done to remove the influence of long loading times or frequent braking of heavy vehicles on performance, and thus create a dataset that for most purposes was homogeneous.

The predicted initial densification was calculated for each site for a time period of one year from construction. The structural deformation model was then used to calculate the predicted rate of rutting progression over the following years. The predicted rates of rutting were then adjusted to those observed on the sites by adjusting the values of the calibration factors for initial densification, K_{rid}, and structural deformation, K_{rst}. The range and average values of these calibration factors are given in Table 3.

The values of the calibration factor, K_{rid}, in Table 3 indicate that the amount of initial densification observed on the sites was less than predicted by HDM-4. For the types of pavements examined during this study, HDM-4 predicts between 2 and 3 mm of initial densification in the first year after construction. The values of K_{rid} derived for these sites indicate that the amount of initial densification observed was approximately 1 mm.

Table 3: Structural Rut Depth Calibration Factors

Route	Initial Densification – Krid			Structural Deformation - Krst		
	avg	min	max	avg	Min	max
Cirebon – Kuningan	0.5	0.2	0.7	1.1	0.7	1.4
Cirebon – Losari	1.0	1.0	1.0	0.9	0.8	1.0
Tangerang – Merak	0.6	0.6	0.6	1.0	1.0	1.0
Kopo – Rancanbali	0.3	0.3	0.3	1.0	0.8	1.2

The values of the calibration factor, K_{rst}, indicate that the rates of structural deformation observed on the sites were generally similar to those predicted by HDM-4. These rates of structural deformation were approximately 1 mm every 4 years.

<u>Plastic Deformation</u>
Five of the sites deteriorated through plastic deformation. For these sites, it was assumed that in addition to plastic deformation, structural deformation had also taken place to the same extent as predicted on the similar sites where no plastic deformation had taken place. Therefore the predicted structural component of rut depth was subtracted from the observed total rut depth and this remaining amount of rutting was assumed to be plastic deformation. The average values of the calibration factors from the non-plastic sites on the relevant route were used for the structural deformation model on the plastic sections.

The values of the variables PT and Sh were fixed at a temperature of 40 °C and an average speed of 60 km/h respectively. The values of the variables VIM, SP and HS were derived from laboratory tests of samples from the sites. The average VIM of ‘plastic’ sites was 2.9% and the average SP was 53 °C.

The observed rates of plastic deformation ranged from approximately 0.5 mm/year to 1.5 mm/year. The predicted rates of plastic deformation were adjusted to these observed rates on the sites by adjusting the calibration factor K_{rpd}. The values of K_{rpd} for the five sites are listed in Table 4. This results in an average K_{rpd} of 1.3 for all sites.

Table 4: Plastic Deformation Calibration Factors

Route	Krpd
Cirebon - Losari	0.7
Cirebon - Losari	1.0
Tangerang - Merak	1.5
Tangerang - Merak	2.0
Tangerang - Merak	1.2

The consequence of plastic deformation on performance, in terms of the mean rut depth, is shown in the Figure 1. It can be seen that a ‘normal’ mix of 4.5% VIM will retain a low rut depth for a considerable number of years, whereas a poorly designed ‘plastic’ mix (with a VIM less than 3%) will deteriorate to high levels of mean rut depth (> 10 mm) within a period of 4 years (VIM = 1%) and 8 years (VIM = 2%). Ensuring quality in design and construction is therefore essential to minimise the need for premature maintenance and reduce costs.

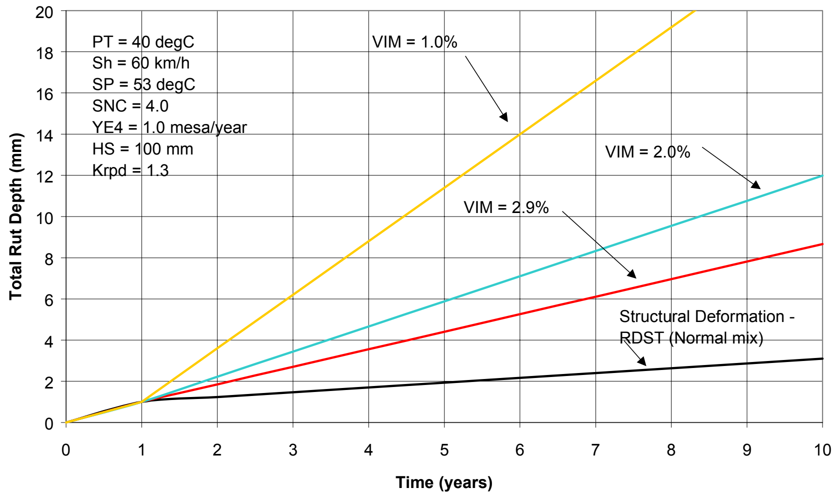

Figure 1: Rut Depth Progressions

4.3 Roughness

The roughness model in HDM-4 is composed of a structural component, surface distress (cracking, rutting and potholing) components and an environmental component. Two calibration factors are used to calibrate the roughness progression model. One factor, K_{gm}, is associated with the environmental coefficient, m, and the other factor, K_{gp}, with the structural and surface distress components.

As described in Section 3.3, the simpler alternatives to the HDM-4 incremental recursive approach were used in this calibration study, by using the detailed (includes distress terms) and the aggregate (no distress terms) models. The calibration of both the detailed and aggregate models was conducted with K_{gm} applied to the environmental coefficient 'm' and K_{gp} applied to the other terms.

The value of the environmental coefficient 'm' was set to 0.023 and the initial roughness, RI_o, was set to 1.7 m/km IRI in this analysis. The range of values for the calibration factors K_{gm} and K_{gp} from both the detailed and aggregate models have been listed in Table 5.

Table 5: Roughness Calibration Factors

Route	Detailed Model						Aggregate Model					
	Kgm			Kgp			Kgm			Kgp		
	avg	min	max	avg	min	max	avg	min	max	avg	min	max
Cirebon – Kuningan	2.5	2.0	4.0	1.0	1.0	1.0	2.7	2.0	4.0	1.0	1.0	1.0
Cirebon – Losari	1.5	1.0	1.8	1.0	1.0	1.0	1.5	1.0	1.8	1.1	1.0	1.2
Cirebon – Ciawi	7.0	4.0	10.0	1.5	1.0	2.0	7.0	4.0	10.0	1.5	1.0	2.0
Tangerang – Merak	1.1	1.0	2.0	1.0	0.6	1.6	4.3	1.0	6.0	1.1	0.8	1.8
Kopo – Rancanbali	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	1.9	1.8	2.0

In the next stage of this analysis, the sites were grouped together by construction quality, terrain and traffic volumes. The average calibration factors for each group have been listed in Table 6.

The figures in Table 6 indicate that for well constructed roads the average calibration factors are approximately equal to 1.0. On poorly constructed roads the calibration factors for the detailed model remain approximately equal to 1.0, but for the aggregate model the values of K_{gm} are high. The reason for this is because the aggregate model does not contain any distress terms and the K_{gm} effectively acts as a proxy for these distress terms.

Table 6: Roughness Calibration Factors by Construction Quality and Traffic

Construction Quality	Annual Traffic (MESA)		Detailed Model		Aggregate Model	
			Kgm	**Kgp**	**Kgm**	**Kgp**
Well constructed roads with average to good asphalt surfacings in flat to rolling terrain and free flowing traffic conditions.	Heavy	> 0.75	1.3	0.9	1.3	1.0
	Light-Medium	< 0.75	2.4	1.2	2.6	1.2
Poorly designed/constructed road, exhibiting failures due to poor road widening and reinstatement prior to overlay and poor mix design, in flat to rolling terrain and free flow traffic conditions.	Heavy	> 0.75	1.3	1.1	5.3	1.0
	Light-Medium	< 0.75	1.0	1.2	5.5	1.4
Well constructed roads located in mountainous regions with average to good asphalt surfacings	All	All	n/a	n/a	n/a	n./a
Well constructed roads in mountainous regions with poor asphalt surfacings	All	All	7.0	1.5	7.0	1.5

It is apparent, from Table 6, that both the detailed and aggregate models are capable of predicting the rate of development of roughness that occurs on well constructed roads, built in flat or rolling terrain, surfaced with well designed asphalt mixes and carrying heavy traffic. When similar roads carry light to medium traffic, both the detailed and aggregate models under-predict the rate of roughness progression. To accommodate this the calibration factor K_{gm}, has to be increased from the default value of one to about 2.5 (2.4 in the case of the detailed model and 2.6 for the aggregate model) to compensate for the extent to which roads still became rough despite the lower values of traffic.

On poorly designed or badly constructed roads, the development of roughness tends to be through the growth of specific defects, such as rutting (both plastic and structural deformation) cracking, potholes and patches. The detailed model was successful at using the extent of these defects to predict the development of roughness. Hence, K_{gm} in the detailed model was close to unity; 1.3 for heavy trafficked roads and 1.0 for light to medium trafficked roads. However, the value of K_{gm} for the aggregate model had to be increased approximately five fold to compensate for the lack of distress terms; ie. 5.3 for heavy trafficked roads and 5.5 for the lighter trafficked roads.

For roads in mountainous terrain, data was only available for roads with poor asphalt surfacings. These failed by either plastic flow or early and very severe cracking. In both cases, one of the major sources of roughness was the development of transverse corrugations. This phenomenon is not considered in either of the models, which both severely under-predict the rate of development of roughness. The values of K_{gm} and K_{gp} were 7.0 and 1.5 respectively, for both the detailed and aggregate models.

The consequences of the above differences are illustrated graphically in Figure 2. On the basis of earlier analysis that it was cost effective to provide an overlay to a highly trafficked road at a roughness intervention level of 4 IRI (Morosiuk, et al, 1999), it is clear that where poor design/construction exists the life of treatments is likely to reduce from approximately 11 years on good construction to 5 years (on flat/rolling terrain) and to 3.5 years (on mountainous terrain).

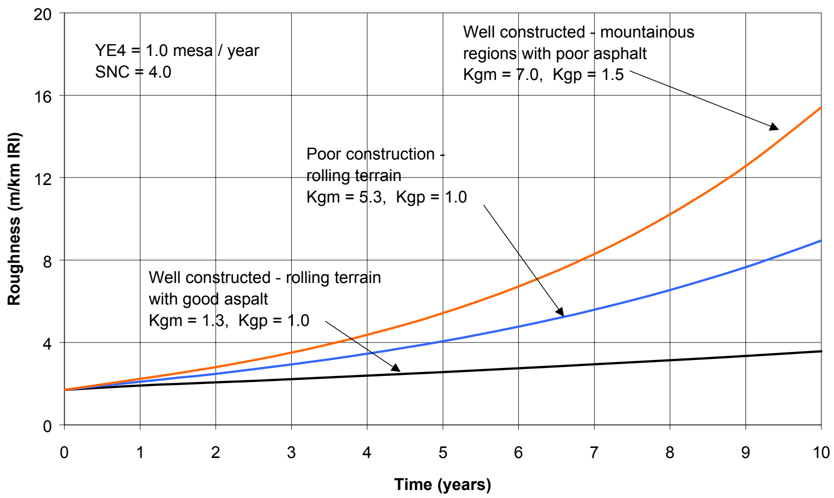

Figure 2: Roughness Progressions

The practical implication of the values of K_{gm} and K_{gp} quoted above is that roads which are badly designed or constructed will develop roughness, and therefore become damaging to vehicles, very much faster then well constructed roads. This is not adequately predicted by the models, especially by the aggregate model that is currently used in the IRMS. If the same value of K_{gm} is used for all roads, assessment of maintenance requirements will be seriously in error. Separate values are required, depending upon:

- changing levels of traffic,
- design and construction quality,
- quality of asphalt surfacing and
- the terrain the road crosses.

5. SUMMARY

The HDM-4 cracking, rutting and roughness relationships were calibrated against observed rates of deterioration of inter-urban roads in West Java. Calibration factors were derived for

- initiation and progression of cracking
- initial densification, structural and plastic deformation components of rutting
- environmental coefficient and progression of roughness

Appropriate use of these calibration factors will enable the HDM-4 deterioration relationships to be used with confidence in pavement management systems in Indonesia such as the IRMS, and for maintenance needs to be more accurately determined. It is also important that road standards and specifications, including those for bituminous overlays, reflect the findings of the study.

6. ACKNOWLEDGEMENTS

The work described in this paper forms part of a programme of research, funded by the Department for International Development (DFID) and the Government of Indonesia, carried out by the Infrastructure and Transportation Department, of TRL's International Division

(Department Head T. Toole) in cooperation with the Institute of Road Engineering (Director Ir Frankie Tayu). Any views expressed are not necessarily those of DFID or the Directorate General of Highways, Indonesia.

7. REFERENCES

ASPHALT INSTITUTE (1983). Asphalt overlays for highway and street rehabilitation. Manual Series No. 17 (MS-17). The Asphalt Institute. College Park, Maryland, USA.

BSI (1985 and 1992). Hot rolled asphalt for roads and other paved areas. British Standard BS 594 Part 1. Specification for constituent materials and asphalt mixtures. London, British Standards Institution.

CORNE C P (1983). Optimising pavement overlay design for Indonesia. Proceedings Fourth Conference of Road Engineering Association of Asia and Australasia, Jakarta, Indonesia.

DACHLAN T, K A ZAMHARI, A B STERLING and T TOOLE (1997). Improved mix design procedures for hot mix asphalts. EASTS '97, Seoul, 29-31 October, 1997.

DARDAK H, T TOOLE, S MAHMUD and T DACHLAN (1992). The performance of asphaltic premix surfacings in Indonesia. 7th REAAA Conference, Singapore, 22-26 June, 1992.

DGH (1986). IBRD Highway Betterment Programme: Specifications for high durability asphalts. Central Design Office. Directorate General of Highways, Jakarta, Indonesia.

MOROSIUK G, A B STERLING and S MAHMUD (1999). Roughness progressions and appropriate maintenance strategies for inter-urban roads in Indonesia. XX1st World Road Congress, PIARC, Kuala Lumpur, Malaysia.

MOROSIUK G, M J RILEY, W D O PATERSON, J B ODOKI and J P COVARRUVIUS (2000). Modelling road deterioration and works effects. Draft Volume 6 of the HDM-4 series of publications. PIARC ISOHDM.

NITRR (1978). Selection and design of hot-mix asphalt surfacings for highways. National Institute for Transport and Road Research. Technical Recommendations for Highways, TRH 8. Pretoria, South Africa.

PATERSON W D O (1987). Road deterioration and maintenance effects. Models for planning and management. The International Bank for Reconstruction and Development, Washington, DC, USA.

PATERSON W D O and B ATTOH-OKINE (1992). Summary Models of Paved Road Deterioration Based on HDM-III. Transportation Research Record 1344, pp 99-105, TRB, National Research Council, Washington, DC.