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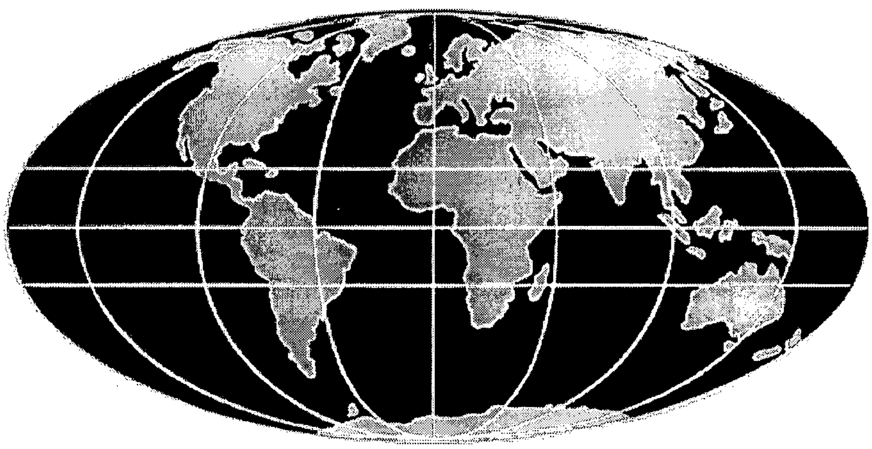

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PERFORMANCE OF ASPHALTIC CONCRETE OVERLAYS IN MALAYSIA

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G Morosiuk, C R Jones and M Hameed

1. INTRODUCTION

A collaborative research programme was established in Malaysia in 1987 between the Transport Research Laboratory (TRL) and the Institute of Public Works of Malaysia (IKRAM) to study ways of improving both the methods of design and construction of new roads, and the design of strengthening and rehabilitation measures for existing roads. One of the projects in this programme examined the performance of asphaltic concrete overlays, constructed as part of the ongoing maintenance programme in Malaysia.

In the mid 1980's, the design of these overlays fell into two categories. Firstly there were maintenance overlays, constructed to a thickness of 40 mm irrespective of the structural capacity of the existing road pavement or the future traffic loading, and are referred to as 'cosmetic' overlays. Secondly, overlays were constructed to a thickness recommended by the design procedure of the Public Works Department (JKR) and are referred to as 'designed' overlays.

Lengths of 2-lane road, generally one kilometre long, were selected as test sections and their performance monitored from construction for up to seven years. The performance of the cosmetic overlays has been previously reported (Rolt et al, 1996). This paper describes the performance of the designed overlays and compares their observed deterioration with that predicted by the deterioration relationships specified in the World Bank's HDM-4 model (Morosiuk, 1999).

The research programme contributed to DFID's aims of increasing the efficiency of national and regional transport systems through the better targeting of scarce resources, thus allowing the potential benefits to accrue to wider sections of the community.

2. EXPERIMENTAL SITES

In order to evaluate the performance of the overlays, lengths of road were required covering a range of pavement strength, pavement condition and traffic level. At the start of the study, several hundred kilometres of road on three routes were identified that were due to be overlaid to a 'designed' thickness. A total of 20 lengths of 2-lane road were selected, with each lane treated as a separate site. The vertical and horizontal geometry of the sites was such that all traffic along the sites was free flowing.

Of the 40 sites, 32 sites on Routes 1 and 12 were monitored from construction of the overlay, and therefore the pre-overlay condition of the surfacings was recorded (including deflections). The overlays on the remaining 8 sites on Route 2 were several years old at the start of the monitoring period, and therefore no pre-overlay information was available. Prior to the construction of these overlays any areas of cracking that were deemed likely to cause reflection cracking were cut out, to a depth of at least 40 mm, and patched with new material. Each site was sub-divided into ten-metre long test areas, referred to as blocks, in each direction of travel. The performance of each block was treated as a discrete unit for the purpose of measurement and analysis.

3. SITE MEASUREMENTS

Surface condition and deflection surveys were carried out on the sites at regular intervals. The condition of the road surface was quantified in terms of cracking and rutting. The severity of cracking within each block was assessed by recording its intensity, width, position and extent, the latter two descriptors then being used to estimate the area of cracking within each block. Rut depths were measured with a 2 metre straight-edge at 10 metre intervals at the beginning of each block.

Pavement deflection tests were carried out on the sites using a Falling Weight Deflectometer (FWD). These tests were carried out at 40 metre intervals in the vergeside wheelpath and the measured deflections were corrected to a standard pressure of 700 kPa and a temperature of 35 deg C. Dynamic Cone Penetrometer (DCP) tests were also conducted on the sites to determine the thickness and strength of the pavement layers.

Traffic volumes were obtained from classified counts conducted by the Highway Planning Unit of JKR. In this study, medium and heavy goods vehicles and buses were classified as commercial vehicles. Axle load surveys were carried out to determine equivalence factors for the commercial vehicles and therefore relate the traffic levels on the sites to the number of equivalent standard axles (esa).

The pavement strengths of the test sites in terms of the modified structural number (SNC) are listed in Table 1, together with the daily traffic flows and the cumulative traffic carried by the sites by the end of the study period.

Table 1
Pavement strength and traffic volumes on the test sites

	Modified Structural Number SNC			**Uni-directional daily traffic flow**			**Total traffic carried (millions esa)**		
	Min	Max	Average	Min	Max	Average	Min	Max	Average
Route 1	4.9	10.4	7.6	3054	7358	5357	9.53	32.28	18.5
Route 12	5.1	9.5	7.2	1473	2575	2038	0.97	6.19	4.0
Route 2	6.4	8.3	7.3	3046	9450	4647	11.10	75.95	29.2

The figures in Table 1 illustrate that the pavements were strong with the average SNC value being in excess of 7.0. Most of the sites on Routes 1 and 2 had carried in excess of 10 million esa by the end of the study. Lower traffic volumes were recorded on Route 12.

4. CONDITION OF THE WEARING COURSE

The overlays were constructed using 14 mm nominal maximum sized aggregate, asphaltic concrete wearing course and the binder content was designed using the 75 blow Marshall method. The designed overlays, generally being in the order of 100 mm thick, included a binder course layer. The JKR specifications (JKR, 1985) for asphaltic concrete material at the time of construction are summarised in Table 2.

Table 2
JKR specifications for asphaltic concrete

Sieve size (mm)	Percentage passing	
	Binder course	Wearing course
25.0	100	-
20.0	78 – 100	100
12.5	60 – 84	78 – 100
10.0	52 – 76	68 – 90
5.0	38 – 62	52 – 72
2.4	28 – 48	38 – 58
0.600	13 – 30	20 – 36
0.300	9 – 22	12 – 25
0.150	5 – 14	7 – 16
0.075	3 – 7	4 – 8
Marshall Properties		
Bitumen content (%)	5.0 – 6.0	5.5 - 7.0
Voids in mix (%)	3 – 7	3 – 5
Voids filled with bitumen (%)	65 – 75	76 – 82
Stability (Kg)	> 500	> 500
Flow (mm)	2 – 4	2 – 4

Samples of the asphaltic concrete tested after construction showed that the aggregate grading of the overlays generally conformed to the JKR specifications.

Visual surveys showed that the performance of the designed overlays, in terms of cracking and rutting, was related to the properties of the wearing course. (Jones et al, 1995). Accordingly each block was assigned a condition category based on the following visual criteria:

- Segregated Areas of obvious segregation of the coarse aggregate
- Coarse No obvious segregation but the surface appears open textured
- Satisfactory Dense surface with rough surface finish
- Good Dense surface with smooth surface finish
- Veining Bitumen veins appearing at the surface but no free bitumen on the surface
- Bleeding Film of free bitumen on the surface of the material

Samples of asphalt in each of the categories were taken and the air voids in the material was measured directly by comparing the theoretical maximum specific gravity (G_{mm}) of the material (D 2041-91 ASTM, 1993) to the bulk specific gravity of the saturated surface dry core (D 2726-93a ASTM 1993). The results, given in Table 3, show that the materials rated as Segregated/Coarse, Good/Satisfactory and Veining/Bleeding had mean air voids, after trafficking, of 7.8, 4.0 and 0.4 per cent respectively.

Table 3
Voids in the mix after trafficking

Quarry	Voids in the Mix (%)								
	Segregated/Coarse			Satisfactory/Good			Veining/Bleeding		
	No.	Mean	SD	No.	Mean	SD	No.	Mean	SD
1	4	9.3	-	19	4.6	1.8	5	0.3	-
2	-	-	-	10	1.0	0.6	2	0.6	-
3	5	6.8	-	3	6.2	-	6	0.3	-
4	3	7.5	-	15	4.7	2.1	5	0.4	-
Total	12	7.8	-	47	4.0	2.3	18	0.4	-

The number of blocks in each condition category, by individual quarry used to supply the asphaltic concrete material, is shown in Table 4. The results show that, in total, only 75% of the overlay material in the study was assessed visually to be in the Satisfactory/Good (G) category. The material supplied from quarry 2 was particularly poor with only 30% being considered satisfactory.

Table 4
Condition of the wearing course material – 7 years after construction

Quarry	Sites	Total No. of blocks	Percentage of blocks		
			Segregated Coarse - C	Satisfactory Good - G	Veining Bleeding - V
1	Route 1 – North	889	5	88	7
2	Route 1 – South	571	38	30	32
3	Route 12	1470	13	77	10
4	Route 2	797	1	81	18
Total		3727	12	75	13

The figures in Table 5 show how the asphalt in the different condition categories has performed in terms of crack intensity and rutting.

Table 5
Cracking and rutting in wearing course material – 7 years after construction

Condition category	No. of blocks	Central Deflection (mm)		Blocks having inter-connected cracks (%)	Blocks having a rut depth of > 5 mm (%)
		Mean	SD		
Segregated/Coarse - C	447	0.31	0.07	52	4
Satisfactory/Good - G	2795	0.31	0.09	30	9
Veining/Bleeding - V	485	0.31	0.09	15	29

The results indicate that significantly more of the material that was Segregated/Coarse (category C) had cracked than the material in the other two condition categories. Similarly, significantly more of the material that was Veining/Bleeding (category V) had rut depths in excess of 5 mm than the material in the other two condition categories. Table 5 also shows that the strength of the road pavement, as measured by the FWD, was similar for each of the condition categories. This indicates that any variation in the development of cracking and rutting was primarily a function of the properties of the surfacing rather than any variation in the structural strength of the road pavement.

5. DETERIORATION OF THE OVERLAYS

To prevent variability in the material properties of the wearing course obscuring the structural performance of the overlays, the blocks were grouped by the three condition categories described in Table 5. These three categories were further sub-divided by areas that had been 'cut and patched' (C&P) prior to overlay and those that had not. The deterioration trends were then examined separately for each group.

5.1 Cracking

The structural cracking relationships in HDM-4 are based on the relationships in HDM-III (Watanatada, 1987). Separate relationships are given for predicting the time to initiation and then the rate of progression. In HDM, crack initiation is deemed to have occurred once 0.5% of the pavement area has cracked.

Structural cracking is modelled as 'all' and 'wide' cracking in HDM. In this study, very little 'wide' cracking had been observed by the end of the monitoring period and therefore only 'all' cracking is reported in this paper.

The relationship for predicting initiation of all structural cracking for overlays is as follows:

$$ICA = K_{cia}\ \max\{4.21\exp[0.14SNC - 17.1(YE4/SNC^2)]\ \max(1 - PCRW/30, 0), 0.025HSNEW\}$$

where

ICA	=	time to initiation of all structural cracking, in years
YE4	=	annual number of equivalent standard axles, in millions/lane
SNC	=	modified structural number of the pavement
PCRW	=	area of wide cracking before overlay, in per cent
HSNEW	=	thickness of latest overlay, in mm
K_{cia}	=	calibration factor for initiation of all structural cracking

The policy of cut and patch used in constructing the 'designed' overlays effectively ensured that the area of wide cracking prior to overlay (PCRW) was zero for all the sites.

The rate of crack progression is modelled incrementally in HDM. The form of the model for the progression of all structural cracking is as follows:

$$dACA = K_{cpa}\, z_a \left[(0.3\, z_a *t_a + SCA^{0.28})^{3.57} - SCA\right]$$

if $ACA_a > 50$, then $z_a = -1$, otherwise $z_a = 1$

where

$$SCA = \min\,[ACA_a,\ (100 - ACA_a)$$

and

dACA	=	incremental change in area of all cracking during time $*t_a$
$*t_a$	=	fraction of year in which all crack progression applies
ACA_a	=	area of all cracking at start of analysis period
K_{cpa}	=	calibration factor for progression of all cracking

The average strength of the pavement (SNC) on the test sites was in excess of 7.0. The HDM crack initiation relationship predicts that cracking in overlaid roads of this strength, carrying 0.5 million esa will occur after 9.4 years, and after 7.9 years on roads carrying one million esa.

The time to initiation of cracks for each condition category of blocks on each site was compared with that predicted by the HDM relationship. In most cases the observed time to crack initiation was less than that predicted by HDM. The predicted times to crack initiation were then adjusted to those observed on the sites by adjusting the calibration factor K_{cia}. The average values of K_{cia} for each condition category are given in Table 6.

The values of K_{cia} show that cracking tended to occur on the 'coarse' sites earlier than for the other two condition categories. As expected, cracking on the 'veining' sites occurred after the other sites.

The HDM cracking progression relationship is a 'S-shaped' curve, with asymtotes at 0 and 100 per cent. The trend of the progression of cracking observed on the sites did not necessarily follow this shape. In calibrating the predicted rate of cracking with that observed on the sites, emphasis was placed on the area of cracking at the end of the study. The values of the calibration factor for crack progression K_{cpa} for each condition category of blocks are given in Table 6.

Table 6
Cracking calibration factors

Condition Category	Number of sites	Initiation Kcia	Progression Kcpa
C - C&P	16	0.36	1.87
C	12	0.40	1.55
All Coarse sites		0.38	1.76
G - C&P	34	0.43	1.36
G	36	0.48	1.21
All Good sites		0.45	1.29
V - C&P	19	0.55	1.12
V	15	0.70	0.46
All Veining sites		0.61	0.94
All sites		0.47	1.33

The values of K_{cpa} in Table 6 indicate that the rate of crack progression observed on the sites was faster than predicted by the HDM relationship, with the exception of the 'Veining' group of sites. The rate of crack progression is highest for 'Coarse' sites and lowest for the 'Veining' sites.

5.2 Rut Depth

In HDM-4 the structural rutting of a pavement is modelled using two components of rut depth; initial densification followed by structural deformation. The initial densification component of rutting is described as rutting that occurs in the first year of a pavement's life and only applies to new construction or reconstruction that involves the construction of a new base layer. In this study only overlays were investigated and therefore the initial densification component was not used in this analysis.

The relationship for progression of structural deformation has been modified in HDM-4 (Morosiuk, 1998) and is given below:

Structural deformation without cracking

$$dRD_{uc} = K_{rd} (44950\ SNC^{-1.14}\ YE4^{0.11}\ COMP^{-2.3})$$

Structural deformation after cracking

$$dRD_{crk} = K_{rd} (2.48x10^{-5}\ SNC^{-0.84}\ YE4^{0.14}\ MMP^{1.07}\ ACX^{1.11})$$

The total annual incremental increase in structural deformation is as follows:

i) before the occurrence of cracking

$$dRD = dRD_{uc}$$

ii) after the occurrence of cracking

$$dRD = dRDST_{uc} + dRD_{crk}$$

where

dRD	=	total incremental increase in structural deformation in analysis year, in mm
dRD_{uc}	=	incremental rutting due to structural deformation without cracking in analysis year, in mm
dRD_{crk}	=	incremental rutting due to structural deformation after cracking in analysis year, in mm
MMP	=	mean monthly precipitation, in mm/month
ACX	=	area of indexed cracking at the beginning of the analysis year, in per cent
YE4	=	annual number of equivalent standard axles, in millions/lane
SNC	=	modified structural number of the pavement
COMP	=	relative compaction of the pavement layers, in per cent
K_{rd}	=	calibration factor for structural deformation

In using the above model to predict the rate of rut depth progression for the test sites, MMP was set to 165 mm/month and COMP to 100 per cent. These rates of progression were then compared with those observed on the sites.

The observed rates of rut depth progression were generally higher than that predicted by the HDM-4 model. The predicted rates were then adjusted to the observed rates by adjusting the calibration factor K_{rd}. The average values of K_{rd} for each condition category are given in Table 7.

Table 7
Structural deformation calibration factors

Condition Category	Number of sites	Krd
C - C&P	16	1.74
C	12	1.25
All Coarse sites		1.60
G - C&P	34	1.70
G	36	1.73
All Good sites		1.72
V - C&P	19	2.41
V	15	2.15
All Veining sites		2.29
All sites		1.95

The values of K_{rd} in Table 7 show that the rate of rut depth progression was highest for the 'Veining' sites and lowest for the 'Coarse' sites. On the 'Veining' sites, the rate of progression was more than twice that predicted by HDM-4 and approximately 30 to 40 per cent higher than the other two groups of sites.

6. SUMMARY

The performance of asphaltic concrete overlays constructed on strong pavements in Malaysia was monitored for a period of seven years and their rates of deterioration, in terms of cracking and rutting, compared with those predicted by the relationships in the HDM-4 model.

The variation in performance of the overlays, in terms of cracking and rutting, was primarily related to the properties of the overlay material as indicated by the voids in the mix after trafficking. The air voids in asphaltic concrete materials are sensitive to both variations in the binder content during manufacture and to the effects of secondary compaction by traffic (Smith and Jones, 1998). Therefore the void content after trafficking is often significantly different to that assumed in the Marshall design. In this study the void content ranged from 7.8%, for those materials considered to be Segregated or Coarse, to 0.4% for those materials characterised as Veining or Bleeding. Materials considered to be either Good or Satisfactory had a VIM of 4% after trafficking.

The study showed that under free flow traffic conditions the rate of crack progression in mixes having VIM of 7.8% was 36% higher than material having voids within normal Marshall specifications. The rate of rut depth progression of material having VIM of 0.4% was 33% higher than material having voids within specified limits.

A combined analysis (All sites) showed that the rates of deterioration of asphaltic concrete overlays constructed to a designed thickness, on sites with free flowing traffic, were generally higher than predicted by HDM-4. It is recommended that the values derived in this study, are appropriate for calibrating the HDM-4 relationships for future use in Malaysia.

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KEYWORDS

MALAYSIA, MAINTENANCE, HIGHWAY DESIGN, MATHEMATICAL MODEL, STRENGTHENING, FLEXIBLE PAVEMENT, ECONOMICS OF TRANSPORT, DEVELOPING COUNTRIES, IBRD.