Optimizing Design Standards for New Pavements Using Highway Design and Maintenance Standards Model (HDM-III)

MIKE J. RILEY, CHRISTOPHER R. BENNETT, DAVID R. SAUNDERS, AND ANDY KIM

A set of new pavement design standards for highways in Thailand derived by using a life-cycle costing approach is presented. The analysis was conducted with the World Bank's Highway Design and Maintenance Standards Model (HDM-III), which simulates the total transport costs of a pavement under different design and maintenance standards. Before the analysis was undertaken, the HDM-III vehicle operating cost and pavement deterioration models were calibrated for Thailand. The analysis was undertaken by first establishing the optimum long-term periodic maintenance policies. This was done by considering a combination of treatments and triggers, with the optimum policy being that which minimized the total transport costs. These periodic maintenance policies were used in conjunction with different initial pavement designs to determine the optimum initial pavement strength by traffic volume. This pavement strength was expressed in terms of the modified structural number. Sensitivity analyses were conducted by using patching only and suboptimal periodic maintenance strategies. These indicated that the current Thailand design standards are appropriate for pavements with traffic of more than 2,000 vehicles per day, given the uncertainties over future maintenance levels and traffic loading. At fewer than 2,000 vehicles per day the current design standards are overdesigning pavements, and a more economical design was recommended. A comparison was made to establish the optimum traffic volume for upgrading asphalt concrete pavements to cement concrete. It was found that unless the time between cement concrete pavement construction and reconstruction is sufficiently long, asphalt concrete pavements are a more economical choice even at higher traffic volumes.

The Highway Design and Maintenance Standards Model (HDM-III) was produced by the World Bank to perform economic appraisals of different road design and maintenance standards (*1*). The program predicts the pavement deterioration and vehicle operating costs over an extended analysis period in which these are dependent on the pavement design standards and the maintenance policies applied to the pavement. It is therefore possible to use HDM-III to determine the optimum new pavement design standards on the basis of economic principles.

This paper presents the results of such an analysis for Thailand. The work was undertaken as part of the Asian Development Bank Technical Assistance Project 1106-THA: Thailand Road Maintenance Project (*2*).

M. J. Riley, Riley Partnership, P.O. Box 177, Kumeu, Auckland 1250, New Zealand. C. R. Bennett, N.D. Lea International. P.O. Box 177, Kumeu, Auckland 1250, New Zealand. D R. Saunders and A. Kim, N.D. Lea International, 251 Consumers Road, Suite 1200, Toronto, Ontario, Canada.

METHODOLOGY

HDM-III Calibration

HDM-III was calibrated for the conditions in Thailand as described previously (*2*). The calibration consisted of modifications to the vehicle operating costs (VOCs) and also changes to the pavement deterioration model. The VOC calibration will be briefly described, and since this paper deals with pavement design standards, the pavement deterioration model calibration will be described in more detail.

A new mechanistic fuel model was supplied for HDM-III. Parameter values for trucks were derived from Thailand data, whereas those for other vehicle classes were based on overseas values. The tire consumption model was modified so that the predictions under average conditions were the same as the observed tire life. On the basis of local studies the Thailand repair and maintenance costs were much lower than those predicted by HDM-III. The HDM-III predictions were reduced by a fixed amount to make the predictions similar to the known values. This approach meant that the roughness effects were not changed from the default effects. The speed prediction model was calibrated on the basis of limited free speed data.

Data on a number of pavement test sections were collected over a 7-year period as part of the Thailand Road Deterioration Study. These data were used to investigate the relevance of the HDM-III pavement deterioration model and to make whatever changes were necessary to it.

It was found that the roughness progression model adequately reflected the observed roughness progression, as did the crack initiation model. However, the rate of observed crack progression was much higher than that predicted by HDM-III. In the Thailand context, HDM-III appeared to be significantly underpredicting crack progression.

HDM-III predicts crack progression only as a function of time. Paterson (*3*) notes:

> The time based models will satisfy most demands of a management system . . . because they represent a good average for typical standards of pavement strengths and loadings. However, when the effects of different loading rates or of pavement strength on cracking are to be evaluated then the traffic based models are preferred.

By using appropriate equations from Paterson (*3*), the traffic-based crack progression models were replaced with time-based ones. Comparison of these predictions with the available crack progression data gave much closer predictions to those observed than the time-based models did. Because the traffic-based model gives unreasonably low predictions for volumes below 1,000 vehicles per day (veh/day), the time-based model was used for volumes below this traffic level.

HDM-III was further modified to take into account the area of previous cracking on postoverlay crack progression. Pavements that have substantial wide cracking before maintenance will suffer from reflective cracking, and thus higher postmaintenance crack progression rates. The annual incremental crack progression was modified by applying the factor

$$\mathrm{INCRFC} = \left(1 + \frac{\mathrm{PCRW}}{100}\right)$$

where INCRFC is a factor to increase the postoverlay crack progression, and PCRW is the percentage of the area with wide cracks before the overlay.

The INCRFC factor indicates that with no wide cracking before the overlay the postoverlay crack progression will be at the standard rate; with 100 percent cracking it will be at twice the standard rate. These rates were not based on any field studies but were selected on the basis of the judgments of the analysts.

A further modification to the pavement deterioration model was in the form of a new equation for predicting the effects of overlays on roughness. A study was conducted at 26 sites in Thailand to investigate the effects of roughness on overlays. Data were collected before and after the overlay by using a TRRL Bump Integrator, calibrated to the international roughness index (in meters per kilometer). The data were analyzed and the following linear equation was developed to predict the impact of 50-mm overlays on roughness (*2*):

$$\mathrm{IRI}a = 1.87 + 0.25\ \mathrm{IRI}b$$

where IRI*a* is the roughness after an overlay (in IRI m/km) and, IRI*b* is the roughness before an overlay (in IRI m/km).

This equation predicts that below 6 IRI m/km an overlay will not reduce the roughness as much as that predicted by the default HDM-III equations (*1*). It was assumed that the relative effects of different overlay thicknesses on roughness embodied in the HDM-III equations were correct, so these effects were used to develop a Thailand model for various overlay thicknesses (*2*). This Thailand model was used to replace the default HDM-III equations.

Analysis Approach

The analysis of alternative pavement designs was undertaken for eight traffic ranges. These ranges were <200, 201 to 500, 501 to 1,000, 1,001 to 2,000, 2,001 to 4,000, 4,001 to 6,000, 6,001 to 10,000, and >10,000 veh/day. The optimum pavement design standard was considered to be that which minimized the total transport costs (i.e., initial construction, periodic maintenance, and road user costs) over a 30-year analysis period.

For flexible pavements the following five surfacing types and thicknesses were evaluated:

- Double bitumen surface treatment (DBST),
- 25-mm asphalt concrete,
- 50-mm asphalt concrete,
- 75-mm asphalt concrete, and
- 100-mm asphalt concrete.

Pavements representing a range of strengths were designed by varying the thickness of different layers (i.e., surface, base and subbase). The strength was expressed in terms of the modified structural number (MSN). This is the traditional AASHO structural number increased to reflect the contribution of the subgrade to pavement strength (*1*).

Pavement Designs

HDM-III simulates the deterioration of a pavement because of the effects of environment and traffic loading. The pavement design parameters used in this simulation are MSN and the thickness of surfacing courses. Of these, MSN largely determines the time required for cracks to appear and the progression of rutting and roughness. Surfacing thickness reduces the rate of cracking through its contribution to pavement strength.

To achieve a specified MSN the least-cost theoretical solution is to put as much of the strength contribution as possible into the lower pavement layers, which, as shown in Table 1, have the highest strength-to-cost ratio.

HDM-III simulates failure because of only aging and fatigue. To guard against shear failure under occasional heavy wheel loads, minimum course thicknesses need to be applied. There are also minimum practical construction thicknesses, normally 100 mm for a granular base or subbase. The thinnest surfacing course is 15 mm for DBST, whereas 150 mm is widely considered to be a safe minimum thickness for a base course. Therefore, the minimum practical MSN for a pavement with a subgrade CBR of 6 percent is 2.21, as shown in Table 2.

The above is based on TRRL Road Note 31 (*4*) and allows some safeguard against heavily loaded trucks on low-volume roads. For higher-strength designs the base thickness should range from 150 to 200 mm and the subbase thickness should range from 100 to 300 mm. Additional select fill should be used to increase pavement strength since the most economical design is to maximize the use of the least-expensive material.

Initial pavement costs vary with subgrade strength since additional material is required to achieve the same MSN. To evaluate the implications of construction on weak subgrades two subgrade strengths were analyzed: a CBR of 6 percent, which is appropriate for most of Thailand, and a CBR of 2 percent, which is found in the soft clay region of central Thailand.

The thickness of various layers required to achieve a specified MSN were established. It was not possible to use all surface types and thicknesses with each MSN because of engineering limitations. Also, some surfacings were unsuitable for certain traffic ranges. The matrix of feasible MSN and surfacing types and thicknesses developed is shown in Table 3.

The cement concrete pavement evaluated was a standard 230-mm concrete slab constructed on a 100-mm base and a 150-mm subbase. Since HDM-III does not contain any cement concrete pavement deterioration relationship, it was decided to simulate the performance of a cement concrete pavement as a strong asphalt concrete pavement (MSN of 7.0) with reduced roughness progression and cracking. Such a pavement was found to have rough-

TABLE 1 Comparison of Relative Costs and Strengths of Pavement Materials

	Asphaltic Concrete	Base	Subbase	Select Fill
Cost (Baht/m^3)[A]	2,500	600	150	100
Cost Index[B]	1.00	0.24	0.06	0.04
Strength coefficient	0.30	0.14	0.10	0.08
Strength index[C]	1.00	0.47	0.30	0.27
Strength/cost[D]	1.00	1.96	5.00	6.75

Notes: A/ 1 $U.S. = 25 Baht.
B/ The cost of each pavement material divided by the asphaltic concrete surfacing cost.
C/ The strength of each pavement material divided by the asphaltic concrete surfacing strength.
D/ The strength index divided by the cost index.

TABLE 2 Calculation of Minimum MSN

Course	Thickness	MSN Contribution
DBST	15 mm	0.18
Base	150 mm	0.84
Subbase	100 mm	0.40
Subgrade Support		0.79
Total MSN		2.21

TABLE 3 Pavement Surfacing Evaluated in Analysis

Traffic Volume in veh/day	Pavement Modified Structural Number						
	2.2	2.5	3.0	3.5	4.0	5.0	6.0
1 - 200	DBST; 25	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50		
201 - 500	DBST; 25	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50		
501 - 1000	DBST; 25	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50	DBST; 25; 50		
1,001 - 2,000		DBST; 25; 50	DBST; 25; 50; 75	DBST; 25; 50; 75	DBST; 25; 50; 75	50; 75	
2,001 - 4,000		DBST; 25; 50	DBST; 25; 50; 75	DBST; 25; 50; 75	DBST; 25; 50; 75	50; 75	
4,001 - 6,000		DBST; 25; 50	DBST; 25; 50; 75	DBST; 25; 50; 75	DBST; 25; 50; 75	50; 75	
6,001 - 10,000			50; 75	50; 75; 100	50; 75; 100	50; 75; 100	75; 100
10,001 - 16,000			50; 75	50; 75; 100	50; 75; 100	50; 75; 100	75; 100

NOTES: DBST - Double Bitumen Surface Treatment
25 - 25 mm Asphaltic Concrete
50 - 50 mm Asphaltic Concrete
75 - 75 mm Asphaltic Concrete
100 - 100 mm Asphaltic Concrete

ness progression similar to that observed on Thailand cement concrete test sections (*2*).

Construction Costs

By using the layer thicknesses from the various pavement designs, the costs of a pavement were calculated by multiplying each layer thickness by the appropriate unit cost.

In Thailand the width of a pavement varies by traffic volume (*2*). The new pavement costs were expressed per meter of carriageway width. Multiplying the per meter costs by the appropriate carriageway width for each traffic volume range produced a series of costs for each traffic volume, surface type and thickness, and MSN. These costs are plotted in Figure 1 for the subgrade with a CBR of 6 percent.

Periodic Maintenance

To determine the optimum initial flexible pavement strength the impact of future maintenance must be taken into account. Thus, the HDM-III evaluations included both the initial design strength of the pavement and the periodic maintenance required over the analysis period.

The optimum periodic maintenance standards were determined by using HDM-III for each of the traffic ranges (*2*).

Since cracking is a major factor influencing overlays in Thailand, it was deemed necessary to trigger overlays as a function of cracking. However, the standard HDM-III triggers overlays only as a function of roughness. The HDM-III source code was therefore modified to include a switch for triggering overlays as a function of cracking, and changes were made to the input routines.

HDM-III was used to determine the total transport costs over a 30-year analysis period in each of the eight traffic ranges. These costs comprised the construction, maintenance, and vehicle operating costs for each pavement surface type-strength combination. By comparing the total transport costs of each pavement for a given traffic volume range the optimum periodic maintenance standards were established. These optimum standards are presented in Table 4. In addition to the optimum periodic maintenance policy, suboptimum and patching only policies were also tested.

Given the competing demands for capital and the insufficient budgets that are available, it is unwise to select projects that are only marginally economically viable. It is not prudent to assume that the VOC estimates are absolutely correct. Although the construction and maintenance costs will be much more accurate, there will still be errors with these costs as well. Thus, were one to select a project with a marginal incremental benefit to cost ratio, there is a possibility that the project is in fact uneconomical.

After assessing various estimates of the accuracy of the VOC and the construction and maintenance cost estimates, the value of 2.5 as a cutoff for the incremental benefit to cost ratio was selected. Thus, the optimum pavement strength was defined as the design that minimized the total transport costs subject to a limiting incremental benefit to cost ratio of 2.5.

Figure 2 is an example of the total transport costs for a pavement with a traffic volume of 2,001 to 4,000 veh/day.

RESULTS OF ANALYSES

Flexible Pavement Strengths

The optimal design standards for both surface treatment and asphaltic pavements are shown in Figure 3 for the eight traffic categories. The optimal design is expressed in terms of the pavement strength by using the MSN.

For low traffic volumes (fewer than 1,000 veh/day) the analyses indicate that the optimum pavement strength is the minimum tested, an MSN of 2.2. With increasing traffic volume the optimum MSN increases steadily up to a value of 4.0 for pavements carrying more than 10,000 veh/day.

The only difference between a pavement with a CBR of 2 percent and one with a CBR of 6 percent is a constant cost increment because of the additional select fill or subbase required to attain the required MSN. Therefore, the optimum MSNs for the two subgrade conditions are the same.

Surface Course Thicknesses

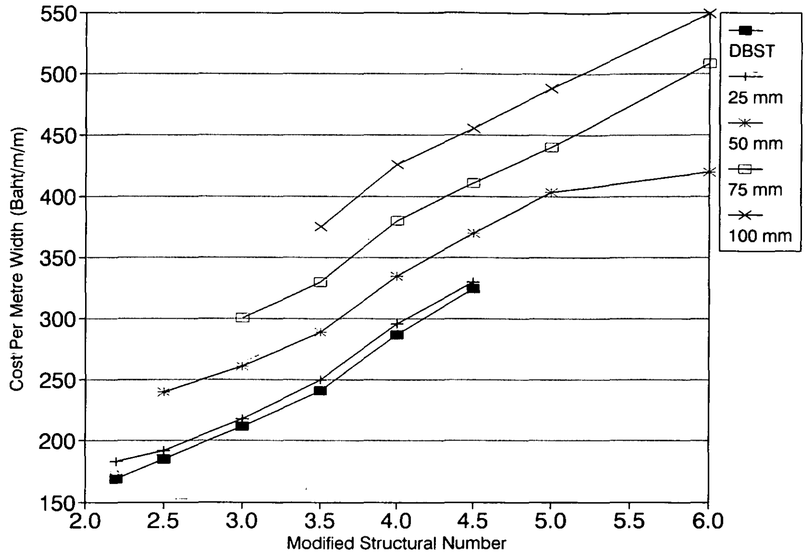
For traffic volumes of fewer than 1,000 veh/day the results indicated that the preferred surfacing is either a DBST or a 25-mm asphalt concrete, with marginal differences between the two. For higher traffic volumes the optimum surface thickness was the thinnest tested for the traffic range. However, HDM-III simulates only the long-term effects of fatigue on surfacing performance and does not allow for shear failure because of small numbers of exceptionally heavy axle loads, loads that are common in developing countries. Therefore, it was recommended that the results of this analysis regarding surfacing thickness should be treated with caution.

When sensible maintenance policies are assumed, the HDM-III pavement deterioration model favors thin surfacings. Consequently, the total transport costs for thin surfacings are less than those for thicker surfacings. The bias toward thin surface courses should therefore be treated with caution. However, it should be recognized that with proper design thin surfaces can offer significant economic benefits over thicker surfacings, particularly for low-volume roads.

Asphalt Concrete Versus Cement Concrete Pavements

The total transport costs of asphalt concrete and cement concrete pavements were compared to determine at what traffic volume roads should be upgraded to cement concrete. The results of this analysis demonstrated the importance of the service life of cement concrete pavements in establishing the volume for upgrading to concrete.

As shown in Figure 4 the total transport costs of constructing a cement concrete pavement are dependent on the length of the time period that extends from when the pavement is first constructed to when it must be reconstructed. For roads forecast to initially carry 6,000 to 10,000 veh/day, a design life of 28 years must be achieved before cement concrete should be selected instead of asphalt concrete. For roads forecast to carry more than 10,000 veh/day initially, the period between reconstruction drops to about 18 years. Thus, unless a suitably long service life can be



NOTES: A/ All costs are economic costs in 1992 Baht ($1 U.S. = 25 Baht)
B/ Costs were calculated by establishing standard designs with combinations of various layer thicknesses to achieve a target MSN. The unit costs of each layer were multiplied by the layer thickness to get the total cost. For each traffic volume, the total cost was divided by the pavement width and these were averaged for all traffic volumes.

FIGURE 1 Pavement costs versus MSN for CBR of 6 percent.

guaranteed, asphalt concrete pavements should be preferred to cement concrete pavements at these volumes.

SENSITIVITY TESTS

The analysis of the optimal flexible pavement design for each traffic range was repeated for two different maintenance policies:

- a suboptimal periodic maintenance policy that over the network would amount to 60 percent of the budget requirements of the optimum standards (Table 4), and
- a minimum maintenance policy of patching only, with no periodic treatments.

For low-traffic-volume roads (fewer than 1,000 veh/day) the results of the suboptimal maintenance policy analysis indicated no change to the optimal designs; that is, a minimum-strength pavement was still the best solution. With higher volumes the optimum MSN increased by about 0.5 compared with the optimum maintenance case.

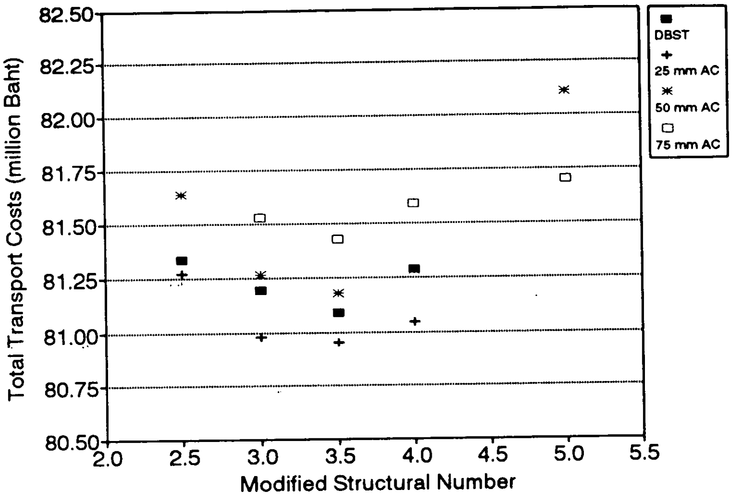
The results of the ''patching only'' sensitivity analysis indicated that the strength requirement was unchanged up to 500 veh/day (i.e., an MSN of 2.2). Thereafter, the required MSN increased rapidly to 6.0 for the highest traffic range. This is illustrated in Figure 3. However, except for very low volume roads, this is an unlikely maintenance scenario for Thailand.

COMPARISONS WITH CURRENT PRACTICE

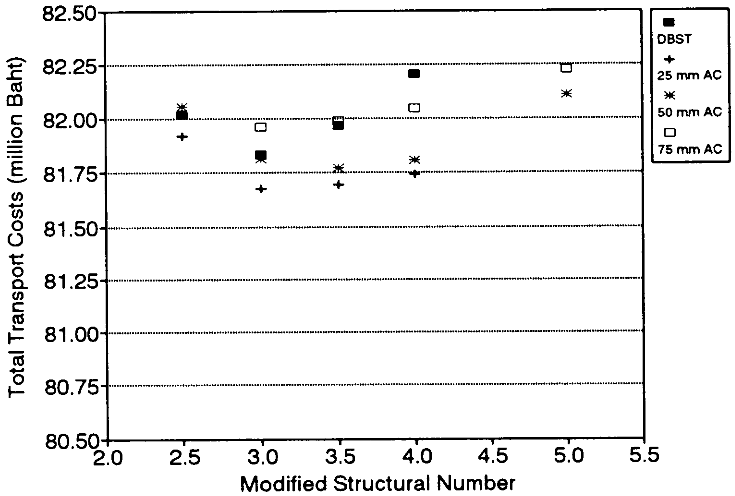
At present the Thailand Department of Highways (DOH) uses the Asphalt Institute (AI) design method and a 10-year design life for

TABLE 4 Optimum Periodic Maintenance Standards

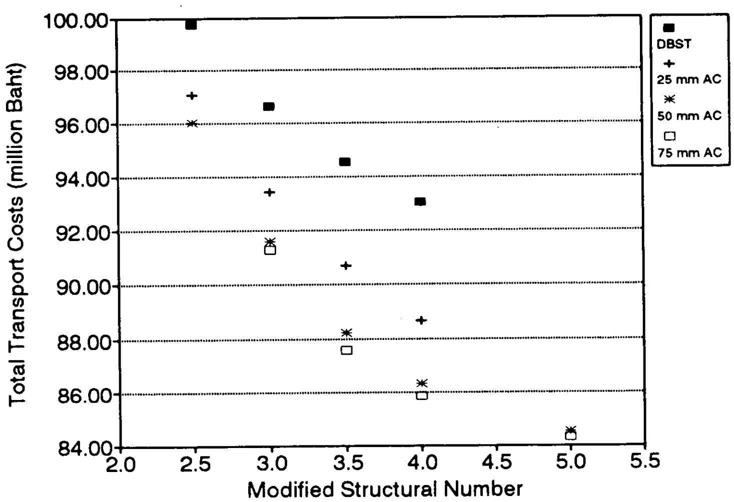
Traffic Volume in veh/day	Reseal at % All Cracks	Overlay at % Wide Cracks	Overlay at IRI Roughness
< 200	10	-	5.0
201 - 500	10	-	5.0
501 - 1,000	10	-	4.5
1,001 - 2,000	10	-	4.0
2,001 - 4,000	10	5	3.5
4,001 - 6,000	10	5	3.5
6,001 - 10,000	10	5	3.5
> 10,000	10	5	3.0



(a)

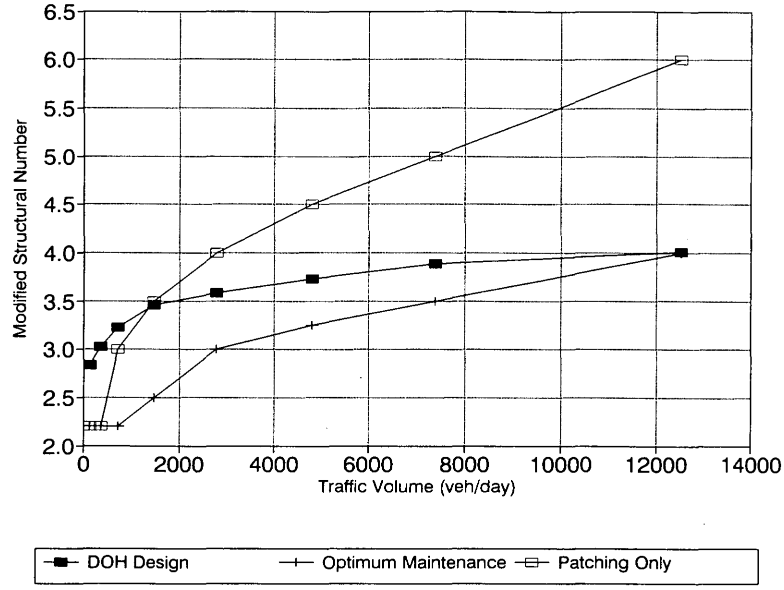


(b)



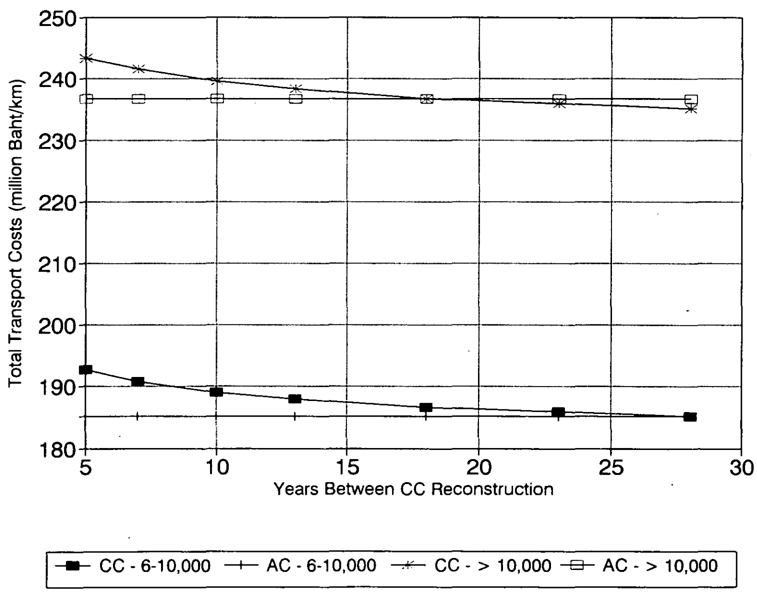
(c)

FIGURE 2 Total transport costs for different pavement designs at 2,001 to 4,000 veh/day: (*a*) optimum periodic maintenance, (*b*) suboptimum periodic maintenance, (*c*) routine maintenance (patching potholes).



NOTES: [A]/ There is no direct conversion between traffic volume and ESAL since the percentage of heavy traffic varied with traffic volume.

FIGURE 3 New pavement strength comparison.



NOTES: [A]/ The term "CC" in the legend pertains to cement concrete; "AC" to asphaltic concrete.

FIGURE 4 Total transport costs of cement concrete (CC) and 50-mm asphalt concrete (AC) pavements.

new pavements. The optimum strengths determined from AI designs are shown in Figure 3, along with the results from the present study for optimal periodic maintenance and patching only. Figure 3 illustrates that there is a large difference between the Thailand optimum with periodic maintenance and the AI design methods at low traffic ranges and that the lines converge at 10,000 veh/day. Making allowance for the risk of substandard maintenance and a future increase in axle loading, it is considered that the present DOH design method is entirely appropriate at traffic volumes of more than 2,000 veh/day.

Below this level it appears that the DOH method produces higher designs than are economically warranted and that the use of TRRL Road Note 31 (*4*) would be more appropriate.

The present practice of using DBST surfacings for roads carrying fewer than 2,000 veh/day is not contradicted by the analyses. For roads carrying traffic above this volume the analyses offer little guidance on surface course thickness for the reasons given earlier.

CONCLUSIONS

This paper has presented the optimum new pavement design standards on the basis of economic principles for highways in Thailand. The analysis was conducted with HDM-III of the World Bank, which simulates the total transport costs of a pavement under different design and maintenance standards.

The economic consequences of different initial pavement designs were established and compared to determine the optimum initial pavement strength. Since the long-term maintenance policies will have a significant impact on the optimal initial pavement design standards the analysis incorporated a regular periodic maintenance policy.

Sensitivity analyses were conducted by using patching only and suboptimal periodic maintenance strategies. These indicated that the current Thailand design standards are appropriate for pavements with traffic volumes of more than 2,000 veh/day, given the uncertainties over future maintenance levels and traffic loadings. At fewer than 2,000 veh/day the current design standards are overdesigning pavements and a more economical design such as that embodied in TRRL Road Note 29 (*4*) is more appropriate.

Unless the time between construction and reconstruction is sufficiently long, asphalt concrete pavements are more economical than cement concrete pavements.

ACKNOWLEDGMENTS

The work described in this paper was funded by the Asian Development Bank. The authors would like to acknowledge the assistance of the Thailand Department of Highways in undertaking the work. This paper is presented with the permission of the Asian Development Bank, Thailand Department of Highways, and N.D. Lea International Ltd.

REFERENCES

1. Watanatada, T., et al. *The Highway Design and Maintenance Standards Model: Description of the HDM-III Model.* World Bank Publications, Washington, D.C., 1987.
2. *Thailand Road Maintenance Project Final Report: Asian Development Bank T.A. No. 1106-THA.* N.D. Lea International Ltd., Vancouver, British Columbia, Canada, 1992.
3. Paterson, W. D. O. *Road Deterioration and Maintenance Effects: Models for Planning and Management.* World Bank Publications, Washington, D.C., 1987.
4. TRRL. *A Guide to the Structural Design of Pavements for New Roads.* Road Note 29, 3rd ed. Her Majesty's Stationery Office, London, 1977.

The views presented in this paper are those of the authors and do not represent the opinions of the Asian Development Bank or the Thailand Department of Highways.

Publication of this paper sponsored by Committee on Flexible Pavement Design.