Optimality of Highway Pavement Strategies in Canada

BRUCE HUTCHINSON, FRED P. NIX, AND RALPH HAAS

The flexible pavement performance predictions of a deterioration model proposed by Small, Winston, and Evans are compared with those estimated from a deterioration model developed for Ontario conditions. The proposed model by Small et al., incorporating even a very modest rate of annual environmentally induced degradation, severely underestimates the initial service lives of Ontario pavements. The life-cycle cost characteristics of Ontario pavements are illustrated, and it is concluded that the optimal pavement strategy is insensitive to changes in the initial pavement thickness. This conclusion is in contrast to the "underbuilt" conclusions by Small et al. Typical average and marginal cost functions that must form the basis of any rational axle weight-based road user charge system are also presented.

Several years ago, Small and Winston (*1*) and Small et al. (*2*) presented an elegant analysis of the optimality of highway pavement structural designs on the U.S. primary highway system. They concluded with respect to optimal pavement durability that

> Our analysis of pavement durability, using standard economic techniques and a new statistical analysis of road test data, suggests that a substantial increase in durability could be achieved at modest cost and would lower the total costs of building and maintaining pavements over their life cycle.

Their conclusion about the optimality of pavement designs was based heavily on their reanalysis of the AASHO Road Test data. They argued that "AASHO's functional specifications and statistical estimation of the coefficients . . . were seriously flawed."

Hudson et al. (*3*) reviewed the analyses conducted by Small et al. (*2*) and performed additional analyses of the performance of the Road Test rigid pavement sections. These new analyses by Hudson et al. included data obtained from the subsequent long-term monitoring of the surviving Road Test sections that had been incorporated into the Illinois Interstate highway network. They concluded that

> the Small/Winston analysis significantly underestimated the life of thick rigid pavements. The original AASHO method overestimated the life of thick rigid pavements, but not as significantly as the Small and Winston survival analysis underestimated the life. Because of the lack of distress in the Road Test for thicker rigid pavements, the Small and Winston survival analysis of AASHO Road Test rigid sections is not valid. (*3*)

Many more of the flexible pavement sections failed at the AASHO Road Test, and the Small and Winston reanalysis of the flexible pavement performance data suggested only relatively small increases in pavement thickness to achieve optimal durability. Small and Winston noted that

> use of the AASHO equations accounts for only about one-third of the difference between optimal and current design (thicknesses); the rest must be due to failure to incorporate economic optimization into design procedure. (*1*)

The purpose of this paper is to highlight some of the findings of a study on road costs conducted for the recently completed Royal Commission on National Passenger Transportation in Canada on road costs (*4*). One of the major questions addressed in this study of Canadian road costs was the "underbuilt thesis" advanced by Small et al. (*2*).

The paper first addresses the issue of pavement deterioration models and compares the behaviors of flexible pavements estimated by a model developed for Ontario conditions with that developed by Small et al. from the AASHO Road Test, which contained some modifications for environmentally introduced pavement deterioration. The second part of the paper analyzes the life-cycle cost characteristics of pavements and identifies optimal pavement strategies. It also partitions the life cycle costs into those caused by vehicle damage and those caused by environmental degradation. The final part of the paper presents representative average and marginal cost functions along with their implications for heavy vehicle pricing.

TYPICAL FLEXIBLE PAVEMENT DESIGNS

Most of the pavements on the major highway system in Canada consist of flexible pavements. Summarized in Table 1 are the characteristics of representative flexible pavement designs for primary highways in three provinces of Canada. The thicker pavements required in New Brunswick reflect the poorer-quality subgrades in that province. Alberta has the smallest range of thicknesses because most of the pavement deterioration is caused by the harsh winter climate rather than by traffic. The thinner base course thickness in Alberta reflects the use of asphalt-stabilized base courses because of the unavailability of good granular base course material. The last row of Table 1 shows the range of equivalent granular thicknesses of the pavement using granular thickness equivalencies for the surface of 2 and for the subbase of 0.67.

PAVEMENT DETERIORATION

In Canada pavement surface quality is usually expressed in terms of the riding comfort index (RCI) rated on a 10-point scale instead of the 5-point scale of the U.S. present serviceability index (PSI). New pavements typically have an initial RCI of 8.5, and first-class highway pavements are normally considered to have deteriorated to an unacceptable condition when the RCI has decreased to 4.5.

B. Hutchinson and R. Haas, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada N2L 3G1. F.P Nix, Orangeville, Ontario, Canada L9W 2Y8.

TABLE 1 Typical Flexible Pavement Structures

Layer	New Brunswick	Ontario	Alberta
asphaltic concrete	140-200	50-130	80-100
base course (granular or asphalt stabilized*)	150	150	50*
granular sub-base	455-760	150-450	180-330
equivalent granular thickness	750-1090	350-700	420-615

entries are in mm

Deterioration Model of Small and Colleagues

Small et al. (2) proposed the following pavement deterioration model on the basis of their reanalysis of the AASHO Road Test data and the incorporation of a term to account for environmental degradation:

$$\mathrm{RCI}(t) = \mathrm{RCI}(0) - [\mathrm{RCI}(0) - \mathrm{RCI}(f)]\left(\frac{Qt}{N}\right)e^{mt} \quad (1)$$

where

- $\mathrm{RCI}(t)$ = RCI at time t,
- $\mathrm{RCI}(0)$ = initial RCI magnitude,
- $\mathrm{RCI}(f)$ = the RCI magnitude at failure,
- Q = the annual number of ESAL coverages;
- N = the total equivalent single axle load (ESAL) coverages expected from a pavement before it deteriorates to the terminal serviceability magnitude RCI (f), and
- m = a parameter to account for the annual decrease in serviceability caused by climatic factors.

The Small et al. (2) reanalysis of the AASHO Road Test data for flexible pavements resulted in the following equation for predicting N, the cumulative ESAL coverages to failure:

$$N = e^{12.062}(D + 1)^{7.761}(L_1 + L_2)^{-3.652}(L_2)^{3.238} \quad (2)$$

where

- D = the structural number of a pavement,
- L_1 = the axle load (in thousands of lbs), and
- L_2 = 1 for single axles and 2 for tandem axles.

Setting L_1 equal to 18 (i.e., the 18,000-lb standard axle load) and L_2 equal to 1, the pavement deterioration model in Equation 1 may be rewritten as

$$\mathrm{RCI}(t) = 8.5 - \left[\frac{4Qt}{3.7021(D + 1)^{7.761}}\right]e^{mt} \quad (3)$$

The value 4 in Equation 3 is the difference between RCI(0) equal to 8.5 and RCI(f) equal to 4.5.

Equation 3 can be used to develop the RCI profile of a representative flexible pavement with a structural number D of 4.9. The magnitude of m equal to 2.3 appears to be representative of the climatic-induced deterioration in Canada from the comments made by Small et al. (2). Small et al. (2) used m equal to 4.0 in most of their calculations, and Paterson (5) suggests that an m in the range of 5.0 to 10.0 might be appropriate for severe climates. There is considerable uncertainty about the appropriate m magnitude, but a value of m of 2.3 is adequate to illustrate the important features of Equation 3.

Illustrated in Figure 1 are the RCI profiles calculated from Equation 3 for D equal to 4.9 for a range of annual ESAL loadings. The diagram shows that the model does not allow for deterioration in the absence of axle loads. The RCI profiles show that the model of Small et al. (2) predicts failure in about 21 years, when the pavement experiences 100,000 ESALs per year, and failure in about 7 years, for an annual ESAL loading of 500,000.

If m magnitudes are used in Equation 3 such as those suggested for harsh Canadian winter conditions by Paterson, say m equal to 7.0, then the terminal serviceability of 4.5 would be reached in about 13 years instead of 21 years for an annual ESAL loading of 100,000.

OPAC Deterioration Model

The Ontario flexible pavement deterioration model (OPAC) provides one of the few models that separates load- from climate-induced deterioration. It was developed from the behavior observed at the AASHO Road Test, the theoretical behavior of layered elastic systems, and the longer-run Brampton Test Road in Ontario (6) Rilett et al. (7) have discussed some of the characteristics of OPAC with respect to cost allocation studies.

The RCI profiles predicted for a flexible pavement with D equal to 4.9 by using the OPAC model are illustrated in Figure 2 for the same range of annual ESAL loadings used in Figure 1. The diagram illustrates that OPAC predicts that pavements deteriorate to an unacceptable condition in about 40 years in the absence of axle loads. With annual ESAL loadings of 1 million, an RCI of 4.5 is reached in about 18 years [versus 4 years for the model of Small et al. (2)];

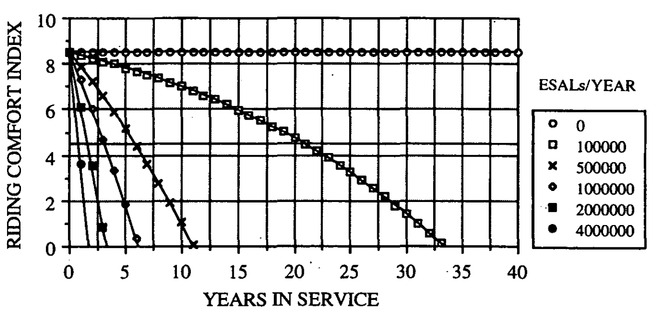

FIGURE 1 RCI versus age profiles from the model of Small et al. (2).

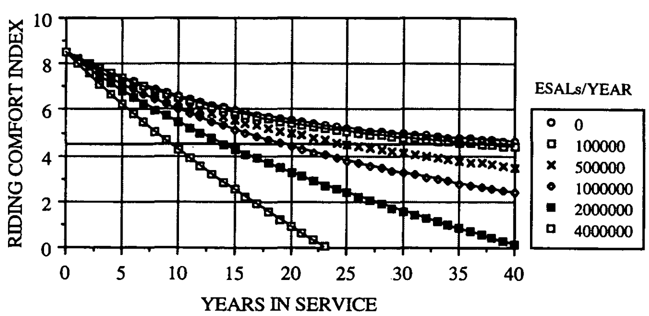

FIGURE 2 RCI versus age profiles from OPAC model.

with an annual ESAL loading of 2 million the terminal serviceability of 4.5 is reached in 14 years [versus about 2 years for the model of Small, et al. (*2*)].

Rilett et al. (*7*) have pointed out that the relative importance of axle loads and environment as sources of pavement deterioration change with different intensities of loads. For example, the environment may account for as much as 80 percent of the deterioration on low-volume roads (fewer than 250,000 ESALS per year) and for only about one-half of the deterioration on heavily loaded roads.

Differences in Failure Life Prediction

Demonstrated in the Table 2 are the differences in pavement lives predicted by the OPAC model and that proposed by Small et al. (*2*). The entries in Table 2 show the initial years to failure for a variety of annual axle load intensities by using both models. The entries confirm the differences between the two models illustrated previously; the model of Small et al. (*2*) forecasts are very sensitive to ESAL loading changes. Clearly, the economically optimum pavement strategy would be much more sensitive to the key design variable, the initial pavement life, if the model of Small et al. (*2*) rather than the OPAC model were used. Small increases in pavement thickness would have substantial payoffs if the model of Small et al. (*2*) were used to predict performance.

ECONOMIC CHARACTERISTICS

The economic characteristics of flexible pavements discussed in this section are based on pavement performance behavior forecast by the OPAC model. The pavement costs are based on unit costs for the surface course of $Can950/mm/two-lane km (US$1 = about $Can 1.35) and for the base and subbase courses of $Can200/mm/two-lane-km.

Illustrated in Figure 3 are the well-known scale economies that exist for highway pavements, in which the present worths of total costs required to achieve a 15-year and a 20-year initial service life to failure are shown as a function of the magnitude of the annual ESAL loading. The pavement performance estimates that form the basis of Figure 3 were obtained from OPAC. Small increases in pavement construction costs can accommodate the substantial increases in ESAL loadings.

Illustrated in the Figure 4 is present value of the construction, resurfacing, and routine maintenance costs for two magnitudes of annual ESAL loadings by using a discount rate of 5 percent. The life-cycle curves are derived from the initial pavement costs, the resurfacing costs required to achieve a 40-year service life (usually two cycles are required), and the routine pavement maintenance costs. The life-cycle cost curves for the high annual axle loadings are quite flat, suggesting that the choice of an initial pavement life is not critical in establishing minimum life-cycle cost thicknesses. For example, in the case of an annual ESAL loading of 2 million, there is only a 2.2 percent difference between the highest and the lowest costs. The life-cycle cost curve for annual ESALs of 250,000 does rise continually, suggesting that the optimal strategy may be to build pavements with short lives. However it should be noted that the costs used in Figure 4 do not include the delay costs to traffic induced by pavement maintenance and reconstruction operations.

Nix et al. (*4*) provide additional analyses of the life-cycle costs of flexible pavements in Canada. They concluded that for the purpose of developing costing procedures for Canadian roads there is no evidence that pavements are being built with less than optimum

TABLE 2 Years in Service Estimated by Two Performance Models

Annual ESAL Load	OPAC Model	Small, Winston & Evans Model		
		m = 0.0	m = 0.023	m = 0.07
100,000	37	34	21	13
500,000	26	7	6	5
1,000,000	20	3	3	3
2,000,000	14	2	2	1
4,000,000	10	1	1	1

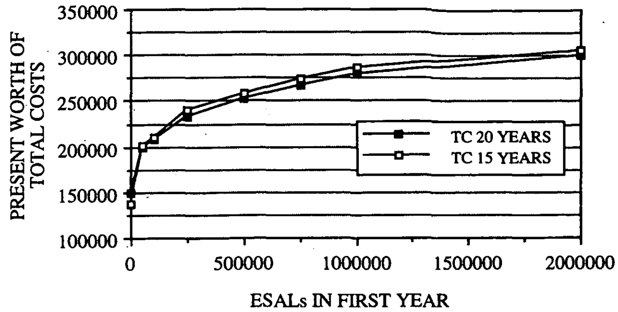

FIGURE 3 Initial construction cost versus annual ESAL loadings.

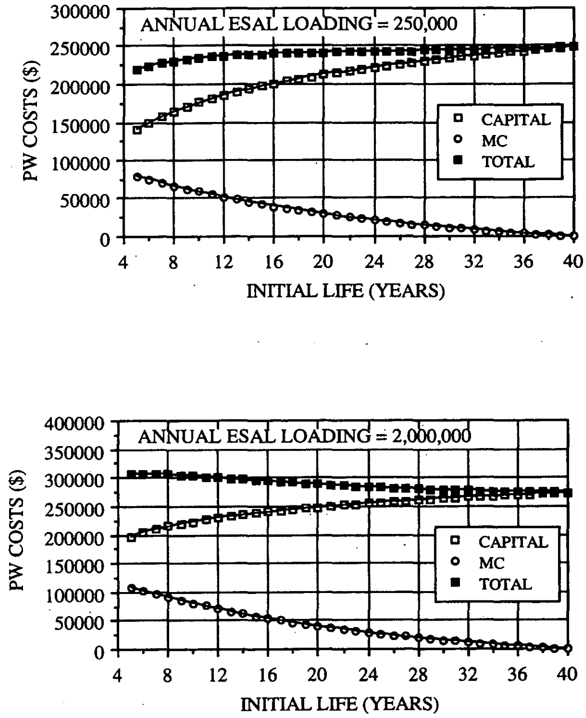

FIGURE 4 Present worth (PW) life-cycle costs versus initial pavement life.

durability. Any initial pavement life of about 15 years seems to be optimal in minimizing total life-cycle costs.

Use of the deterioration model of Small et al. (*2*) as the basis for the economic analysis would yield very different conclusions about the optimality of flexible pavement designs in Ontario. Total life-cycle costs would be higher, and the rate of change of costs with initial pavement costs with changes in initial pavement life for each level of ESAL loading would be higher.

Illustrated in Figure 5 is the present worth of the life-cycle pavement costs for an initial pavement life of 15 years for different annual ESAL loading magnitudes. The following function has been fitted to the data:

$$C = 89{,}969 + 23{,}214 \log (\text{ESALs}) \tag{4}$$

where C is the present worth of the life cycle costs.

Illustrated in Figure 6 is the marginal pavement cost (MC) function derived from Equation 4, which has the following equation:

$$\text{MC} = \frac{23{,}214}{\text{ESALs}} \times \frac{1}{\ln 10} \tag{5}$$

The average cost function is also shown in Figure 6, and because of the scale economies it falls above the marginal cost function.

The implications of these marginal cost functions for three representative Ontario truck types and for three ESAL volumes by using two methods for converting axle loads to ESALs are illustrated in Table 3. The upper number in each cell uses the AASHTO ESAL equivalencies, whereas the lower number uses the functions

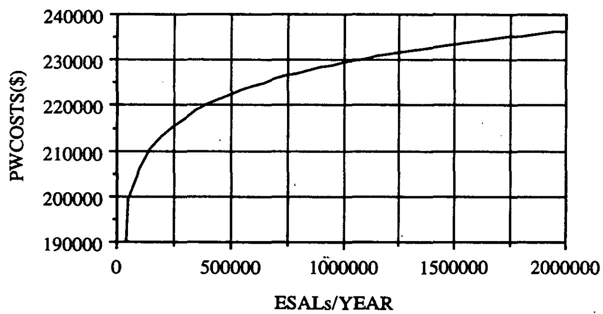

FIGURE 5 Present worth of life-cycle costs versus annual ESAL loadings.

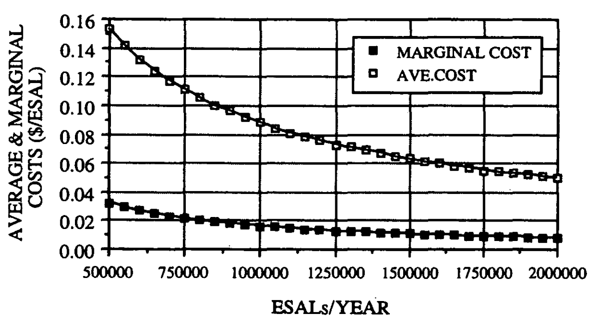

FIGURE 6 Marginal and average pavement damage costs per ESAL versus annual ESAL loadings.

TABLE 3 Marginal Pavement Costs for Several Ontario Truck Types

Truck Type	Annual ESAL Load Intensity		
	Low (50,000)	Medium (500,000)	High (2,000,000)
3 axle (25 t)	10.3 19.5	2.6 4.9	1.3 2.4
5 axle tractor-semi (39 t)	13.5 23.4	3.4 5.8	1.7 2.9
8 axle B-train (62 t)	18.6 33.1	4.6 8.3	2.3 4.1

entries are cents per kilometre

reported by Rilett and Hutchinson (*8*). The marginal costs listed in Table 3 are for the traffic volumes listed at the head of each traffic volume column and are calculated from Equation 5.

The Table 3 entries show the large variations in the marginal costs per kilometer between truck types and across road types, for example, \$0.186/km for a 3-S3-S2 operating on a road with low annual ESAL loadings versus \$0.023/km for the truck when it operates on a road with high annual ESAL loadings. Charging trucks for load-associated pavement damage at the long-run marginal cost would not recover total pavement damage costs, and a second charge would be required to recover fully the remainder of the damage costs, the nonload-associated costs, and the other costs associated with the provision of highway infrastructures.

CONCLUSIONS

The initial service lives of flexible pavements forecast by the model of Small et al. (*2*) containing a modest annual environment-induced degradation are much shorter than those observed in Ontario. The more rapid decrease in RCI estimated by the model of Small et al. (*2*) is probably caused by its use of the AASHO Road Test data and the accelerated loading of the pavements in this Road Test.

Analyses of the life-cycle costs of flexible pavements by the OPAC degradation model for a range of annual ESAL loadings showed that weak minima existed. This means that optimal pavement strategies in Ontario are not sensitive to the issue of initial pavement durability (thickness) as suggested by Small et al. (*2*).

The strong economies of scale present in flexible pavements produce long-run marginal pavement costs per ESAL that are much lower than the long-run average costs. This means that ESAL-kilometer charges for loaded trucks based on long-run marginal costs would not fully recover life-cycle costs. Other mechanisms such as Ramsey pricing, higher vehicle registration fees, charges for externalities, and charges for capacity consumption would be required to recover the additional costs of providing highways.

REFERENCES

1. Small, K. A., and C. Winston. Optimal Highway Durability. *The American Economic Review,* Vol. 78, No. 3, 1988, pp. 560–565.
2. Small, K. A., C. Winston, and C. A. Evans. *Road Work: A New Highway Pricing and Investment Policy.* The Brookings Institution, Washington, D.C., 1989.
3. Hudson, W. R., M. T. McNerney, and T. Dossey. A Comparison and Re-Analysis of the AASHO Road Test Rigid Pavement Data. Presented at 70th Annual Meeting of the Transportation Research Board, Washington, D.C. 1991.

4. Nix, F. P., M. Boucher, and B. G. Hutchinson. Road Costs. *Directions,* Vol. 3. Final Report. Royal Commission on National Passenger Transportation, Ottawa, Ontario, Canada, 1992, pp. 937–1058.
5. Paterson, W. D. O. *Road Deterioration and Maintenance Effects: Models for Planning and Management.* The World Bank, The Johns Hopkins University Press, 1987.
6. Jung, F. W., R. Kher, and W. A. Phang. *A Performance Prediction Subsystem—Flexible Pavements.* Research Report 200. Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1975.
7. Rilett, L. R., B. G. Hutchinson, and R. C. G. Haas. Cost Allocation Implications of Flexible Pavement Deterioration Models. In *Transportation Research Record 1215,* TRB, National Research Council, Washington, D.C. 1989, pp. 31–42.
8. Rilett, L. R., and B. G. Hutchinson. LEF Functions from Canroad Pavement Load-Deflection Data. In *Transportation Research Record 1196,* TRB, National Research Council, Washington, D.C. 1988, 170–178.