

Effect of Performance Model Accuracy on Optimal Pavement Design

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Abstract: *In the first part of this paper, an analysis of the data collected during the American Association of State Highway Officials (AASHO) road test, based on probabilistic duration modeling techniques, is presented. Duration techniques enable the stochastic nature of pavement failure time to be evaluated as well as censored data to be incorporated in the statistical estimation of the model parameters. The second part of this paper presents the use of economic optimization principles for determining the optimal design of flexible pavements. We study the effect of deterioration model accuracy on optimal design and lifecycle costs by comparing three models. The first is a simple regression model developed by the AASHO, which forms the basis of design standards in use today. The second is a regression model that was developed with the same AASHO data set, but that includes a correction for data censoring. The third model is the probabilistic model developed in the first part of this paper. The results show that the AASHO model, when used as an input to lifecycle cost minimization, produces a pavement structural number that is lower than that produced by using the other two deterioration models. This results in shorter pavement lives and higher costs due to more frequent resurfacing. The savings in lifecycle cost accrued by using optimal structural number are shown to be quite significant, offering a sound basis for revising current design practices.*

1 INTRODUCTION

Predicting the actual performance of a specific pavement section under the combined action of traffic and environmental conditions provides valuable information to a

highway agency for proper work planning and budget allocation. Pavement failure is a highly variable event that not only depends on layer material properties, environmental and subgrade condition, and traffic loading, but also on the specific definition of failure adopted by the highway agency. Failure can be defined in terms of amount of cracking, rut depth, surface roughness, or combinations of these or other indicators of performance.

Accelerated pavement testing (APT) has been used in many countries to evaluate pavement response and performance and to establish failure models for pavement life predictions. The best among these models have incorporated some kind of mechanistic-empirical considerations of the failure process. Although these mechanistic-empirical approaches include some stochastic considerations of material properties, environmental conditions, and/or traffic loading, their predictions are in general deterministic and, therefore, unrealistic. The first full-scale APT experiments were conducted in the United States in the late 1950s, and they consisted of specially built sections subjected to actual traffic loading. One of these tests deserves special mention because of the influence it exerted on pavement design: the AASHO road test (AASHTO, 1986).

1.1 The AASHO road test

The AASHO road test took place in the late 1950s and was located near Ottawa, Illinois. The site was chosen because the soil in the area was uniform and representative of soils in large areas of the country. The climate was also considered to be representative of many states in the northern United States (HRB, 1960). Hence, only one subgrade material was evaluated during the experiment as well as only one climatic region. Even though both conditions are typical of large areas of the United States, the use of

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the results outside these conditions should be subjected to detailed assessment of their applicability. Besides, estimation of the effects of other subgrade material and/or environmental conditions cannot be attained with this data set.

The test tracks consisted of four large loops, numbered 3 through 6, and two small loops, numbered 1 and 2. Each loop was a segment of a four-lane divided highway whose north tangents were surfaced with asphalt concrete (AC) and the south tangents with Portland cement concrete (PCC). Only loops 2 through 6 were subjected to traffic, and all vehicles assigned to any one traffic lane had the same axle load configuration. Whenever possible, the traffic was operated at 56 kph (35 mph) on the test tangents. A total of approximately 1,114,000 axle load repetitions were applied from November 1958 until December 1960.

Most of the sections on the flexible pavement tangents were part of a complete experimental design, where the design factors were surface thickness, base thickness, and sub-base thickness. The dimensions of the main factorial designs were $3 \times 3 \times 3$, that is, three levels of surface thickness combined with three different base thicknesses and three sub-base thicknesses.

The materials used for the construction of the AC surface, base, and sub-base layers were kept the same for all sections; hence, the effect of the material properties on pavement performance cannot be assessed from the data of the main experimental design. Other experiments aimed at assessing different surface and base materials were also conducted during the AASHO road test but were not part of the main experimental design and, therefore, were not considered in the development of the models presented in this paper.

2 LITERATURE REVIEW

2.1 Regression models

The initial attempts at developing performance models used regression analysis to estimate pavement damage functions. Damage functions are mathematical equations aimed at predicting a specific distress, response, or reduction in functional or structural performance as a function of traffic loading or time. The first form of such function was developed based on the analysis of the data of the AASHO road test. The form of the damage function was the following:

$$g_t = \left(\frac{ESA_t}{\rho} \right)^\beta \quad (1)$$

where g_t = damage at time t ,
 ESA_t = number of equivalent standard 18 kips
 axle loads applied up to time t ,
 ρ = ESA required to produce a damage
 level defined as failure, and

β = power that represents the rate of
 damage increase.

ESA_t represents the equivalent traffic at some time of interest t before failure is reached. The parameters ρ and β (estimated by regression analysis) differ with the type of distress and are functions of a variety of explanatory variables consistent with the form of performance under consideration. In the AASHO design equation, the damage function is defined in terms of the serviceability index ratio:

$$g_t = \frac{p_o - p_t}{p_o - p_f} \quad (2)$$

where p_o and p_f are the initial and terminal conditions as measured by the serviceability index (PSI), and p_t is the value of PSI at time t . Based on the above definition of damage, the form of the AASHO performance model was:

$$p_t = p_o - (p_o - p_f) \left(\frac{ESA_t}{\rho} \right)^\beta \quad (3)$$

The damage function given in Equation (1) resulted from accelerated testing in one environment and essentially for one subgrade, so new damage functions applicable to other environments and subgrades were needed. Research incorporating mechanistic principles and new experimental data developed improved models for the prediction of the reduction in PSI, rut depth, and fatigue cracking (Rauhut et al., 1983). These researchers developed models that could be applied to a wider range of conditions than those of the AASHO road test.

Another improvement to the original AASHO model was due to the observation that an S-shaped curve would predict more realistic long-term pavement performance (Garcia-Diaz & Riggins, 1984). That is, it was recognized that the pavement deterioration rate decreases as the end of the service life of the pavement is approached. This behavior is typical of pavements that have received adequate routine maintenance in the past.

A similar basic approach has been used more recently (Sebaaly et al., 1995) to develop nine performance models for flexible pavement maintenance treatments. It was concluded that the nine models developed have very good fit to data as measured by the coefficient of determination (R^2) of the regressions. It was also established that the sign of the variables' coefficients were sometimes opposite to common belief. This situation was attributed to the presence of outliers within the data set, without further analysis of the possible statistical significance of these outliers. However, it is possible that some fundamental errors were committed during the statistical analysis of the data. Because each data set consisted of pavement sections that received a specific treatment, and because the treatments were not assigned randomly by the highway agency, it is likely that these data

were selected and that the resulting models suffered from selectivity bias (Madanat and Mishalani, 1998).

Other authors have established that AASHO's functional specification and statistical estimation of the coefficients in the deterioration model were seriously flawed, due to inappropriate treatment of censored observations. They proposed new estimates based on Tobit analysis (Small and Winston, 1988). The latter study also established that the original AASHO models overestimate the life of thicker pavements (those with large structural number) and concluded that this is a possible reason for higher traffic highways not performing as expected. It should also be recognized that prediction of the estimated life for thick pavements (based on the AASHO road test data) involves extrapolation well beyond the range of direct observation. Although several researchers have proposed improvements to the AASHO equation, these improvements never enjoyed the widespread applicability that the original equation did.

2.2 Duration models

The initiation of pavement distress is a highly variable event, that is, distress occurs at different times at various locations along a homogeneous piece of road. Hence, the time of failure should be represented by a probability density function rather than by a point estimate (deterministic estimation). Because data collection surveys are in many cases of limited duration, in addition to the considerable variability in failure times, there is the difficulty of unobserved failure events in a typical set of pavement condition data (Paterson, 1987). Often only the events observed during the survey are included in the statistical analysis. Therefore, important information about the stochastic and mechanistic properties coming from before and after events are excluded causing a bias in the model. These excluded events are known as censored data (Greene, 1993).

Both the stochastic variations and the censoring of the dependent variable can be addressed by developing an estimation procedure based on the principles of failure-time analysis (Paterson and Checher, 1986; Paterson et al., 1989). The procedure makes use of the following:

1. The statistical method of maximum likelihood estimation to exploit both censored and uncensored data, and
2. A distribution that enables the variability of failure times to be determined from the data. A Weibull distribution was selected for this purpose.

Due to the randomness of pavement distress and the fact that information is only available over a limited period, duration models accounting for censored data seem to be the better approach to deterioration modeling. This is especially the case when experimental pavement data are used, because such data sets consist of continuous observations

of pavement performance, allowing the analyst to obtain precise measurements of failure times.

3 MODELING APPROACH

In this section, we introduce the modeling approach used in this paper. We will present the approach in the context of failure time for simplicity. However, it should be remembered that our variable of interest is the number of equivalent standard axle load repetitions to failure, rather than failure time.

As indicated earlier, terminal pavement conditions will be reached at different times at various locations along a homogeneous road. If we refer to this time as τ , we can consider τ as a random variable with a given density $f(\tau)$. In general, pavement engineers are interested in the probability that the pavement will not have failed by a certain age t ; this is represented by a survival function $S(t)$. By definition, $S(t) = Pr(\tau > t) = 1 - F(t)$, where $F(t)$ is the cumulative distribution function of τ .

A common problem in modeling event duration is caused by unobserved failure events in a typical data set. Data collection surveys are usually of limited length and, in general, they do not start at the beginning of the pavement life. That is, for a given data set, some pavement sections will have already failed by the day the survey starts, others will reach terminal conditions during the survey period, while others will only fail after the survey is concluded.

If only the failure events observed during the survey were included in the statistical analysis (disregarding the censored information after and before events), the model developed would suffer from truncation bias. On the other hand, if the censoring of the failure events is not accounted for properly, then the model may suffer from censoring bias. The information on the "after" events is of particular importance in representing the stronger, long-life pavement sections. By using probabilistic duration techniques, both the stochastic variation of failure times and the potentiality of censored data are incorporated in the development of the model (Kalbfleisch and Prentice, 1980).

3.1 Censored data

The random variable τ will be defined as the time from initial construction of the pavement section until a preset terminal condition level is reached. If the collection data survey extends from t_0 years after construction (or rehabilitation) until t_f , then one and only one of the following three conditions can be observed:

1. $\tau \leq t_0$. In this case the pavement has failed prior to the initiation of the data collection survey. In this case, we define $Y_1 = 1$, otherwise $Y_1 = 0$.

2. $t_0 < \tau \leq t_f$. This condition implies that failure has occurred during the observation period. In this case we define $Y_2 = 1$, otherwise $Y_2 = 0$. We also define t as the observed value of τ .
3. $t_f < \tau$. In this case failure has not occurred by the time the data collection survey concluded. In this case, we define $Y_3 = 1$, otherwise $Y_3 = 0$.

For conditions (1) and (3) above, we do not know the actual failure time, so we need to define a new random variable T :

$$\begin{aligned} T &= t_0 & \text{if } Y_1 &= 1, \\ T &= t & \text{if } Y_2 &= 1, \end{aligned}$$

and

$$T = t_f \quad \text{if } Y_3 = 1.$$

By taking a sample of pavement sections, we obtain values for Y_1, Y_2, Y_3 , and T and consider the joint probability in order to develop a maximum likelihood estimator. The joint probability consists of the product of the probability mass function of the discrete variables Y_1, Y_2 , and Y_3 and the probability density function of the continuous variable T . Multiplying marginal and conditional probabilities, we obtain the joint probability for each section:

$$P(Y_1 \cap Y_2 \cap Y_3 \cap t) = F(t)^{Y_1} f(t)^{Y_2} S(t)^{Y_3} \quad (4)$$

We can develop a pavement failure model by expressing the conditional failure time in terms of a vector of exogenous variables X (such as traffic loading and pavement structure attributes) and a vector of parameters $\underline{\theta}$ as follows:

$$\begin{aligned} P(Y_1 \cap Y_2 \cap Y_3 \cap t | \underline{X}, \underline{\theta}) \\ = F(t | \underline{X}, \underline{\theta})^{Y_1} f(t | \underline{X}, \underline{\theta})^{Y_2} S(t | \underline{X}, \underline{\theta})^{Y_3} \end{aligned} \quad (5)$$

The estimation of $\underline{\theta}$ can be achieved by using maximum likelihood estimation. Further details of the estimation procedure can be found in the work by Prozzi and Madanat (2000).

3.2 Variability of failure time

The time to failure of a given pavement section depends also on the definition of failure used. In the AASHTO road test, a pavement section was considered to have failed when its serviceability index reached a value of 2.5. Our discussion of failure is given in this context.

The hazard rate function $\lambda(t)$, a concept used in reliability theory, is proportional to the probability that failure will occur in a short time interval given that it has not occurred previously. It is defined as:

$$\lambda(t) = \frac{f(t)}{1 - F(t)} = \frac{f(t)}{S(t)} = -\frac{d \ln S(t)}{dt} \quad (6)$$

Hence, the hazard rate is the rate at which failure will occur after a time t , given that it has not happened yet at time t . For our purposes it may be preferable to model the hazard function rather than the density function, the cumulative density function, or the survival function. In any case, all four are related as indicated in the previous equation.

As indicated previously, it seems more appealing to model the hazard function directly, rather than the survival function and then, for estimation purposes, differentiate to obtain the density. In the case of pavements, it could be argued that the hazard function is increasing or decreasing. However, it is likely that the hazard function increases (i.e., that the rate at which failure occurs increases with pavement age).

Therefore, a variable rate hazard function that results in a Weibull distribution was used in this study. In this case, the rate can either increase or decrease depending on the value of the parameter p , ($p \neq 1$) that is estimated from the data. The Weibull hazard function is given by:

$$\lambda(t) = \lambda p (\lambda t)^{p-1}, \quad \text{hence} \quad S(t) = e^{-(\lambda t)^p} \quad (7)$$

The parameters $\underline{\theta} = (\lambda, p)$ of this model can be estimated by maximum likelihood.

The parametric model described above can be extended to account for the effect of exogenous factors in shaping the survival distribution. In the case of the prediction of pavement failure time, the extension of the Weibull model will be considered:

$$\lambda_i = e^{-\underline{\theta} \underline{X}_i} \quad (8)$$

where $\underline{\theta}$ is a vector of parameters and \underline{X}_i is a vector of exogenous variables for pavement section i , which are assumed not to change from $T = 0$ to the failure time $T = t$. Making λ a function of a set of exogenous variables is equivalent to changing the units of measurement on the time axis. For this reason these models are sometimes called “accelerated failure time” models.

The hazard function depends on t, p , and X . The signs of the estimated coefficients suggest the direction of the effects of the variables on the hazard function when the hazard is monotonic. In the case of the Weibull model, the expected duration is easily computed as:

$$E[t | \underline{X}_i] = \exp(p \underline{\theta} \underline{X}_i) \quad (9)$$

4 APPLICATION

In section 2, we presented the form of the selected deterioration model developed during the analysis of the AASHTO road test data. However, these equations do not account for the influence of pavement and traffic characteristics on pavement performance. In order to account for the influence of exogenous variables, it was proposed that the values of β (deterioration rate) and ρ (number of repetitions

to failure) should be expressed as a function of pavement and traffic characteristics. The resulting equation for ρ was (HRB, 1960):

$$\rho = \frac{A_o(D+1)^{A_1}L_2^{A_3}}{(L_1+L_2)^{A_2}} \quad (10)$$

where D = structural number of the pavement section,
 L_1 = axle load in kips,
 L_2 = dummy variable ($L_2 = 1$ for single axles; $L_2 = 2$ for tandem axles),
 A_o, \dots, A_3 = parameters.

The parameters of the above equation were estimated using an ad-hoc stepwise regression procedure. The following equation was developed:

$$\rho = \frac{10^{5.93}(D+1)^{9.36}L_2^{4.33}}{(L_1+L_2)^{4.79}} \quad (11)$$

Further details of how the parameters of this equation were estimated are unclear in the literature. In particular, it is uncertain how the censored data were accounted for. Despite all its known flaws, this equation (or slight variations of it) has been the main tool for the design of bituminous roads in the U.S. and around the world for almost half a century. It is also well known that a number of studies have reported great variations between the predictions of the above equations and the performance of actual pavements (Paterson, 1987; Small and Winston, 1988).

In our research, we used duration models to develop a counterpart to the above equation. In the context of the AASHO road test data, the observation period started with new pavement sections. Therefore, none of the pavement failures occurred prior to the observation period and thus we did not face the problem of left censoring. On the other hand, several sections had not reached failure at the end of the experiment. Thus, we had a number of "after" events, which are right-censored observations.

We used the Weibull model, in which the hazard rate was a function of exogenous variables. These variables were the regressors used in the AASHO model, namely D , L_1 and L_2 . The results of the estimation are presented in Table 1.

The resulting equation (in terms of equivalent standard axle load repetitions) is:

$$E[\rho] = \exp(12.15 + 6.68 \ln(D+1) + 2.62 \ln(L_2) - 3.03 \ln(L_1 + L_2)) \quad (12)$$

which, transformed into a similar form as the AASHO equation, results in the following expression:

$$E[\rho] = \frac{10^{5.28}(D+1)^{6.68}L_2^{2.62}}{(L_1+L_2)^{3.03}} \quad (13)$$

Table 1
Estimation results

| Variable | Estimated Parameter | t Statistic |
|----------------|---------------------|-------------|
| Constant | 12.15 | 50.93 |
| $\ln(D+1)$ | 6.68 | 37.03 |
| $\ln(L_1+L_2)$ | -3.03 | -23.47 |
| $\ln(L_2)$ | 2.62 | 12.30 |
| σ | 0.75 | 17.62 |

Dependent variable number of equivalent axle loads to failure (ρ).
Number of observations 284.

It can be seen that the parameter estimates in Equation (13) are smaller in magnitude than the corresponding estimates in Equation (11). The predictions of pavement lives obtained by using this new equation match the observed lives better than those obtained by using the original AASHO equation for the same set of explanatory variables. This can be objectively measured by the estimates of the standard errors of the two models, which are given by the square root of the average squared residuals. The standard error of the AASHO equation is 0.65, while the standard error for our equation is 0.42, representing a reduction in the standard error of the forecast of about 35%. Thus, the new equation is not only statistically sound but also fits the data better than the original AASHO equation. Additional details about this model are provided in Prozzi and Madanat (2000).

The exponent of the variable $(L_1 + L_2)$ is of particular interest, since it indicates the sensitivity of the pavements to overloading. Hence, it is an indicator of the damage coefficient of the well-known power law. It should be noted that the new exponent is 36% lower than the original one. Because this exponent has a significant influence on the design of heavy traffic pavements, this difference is expected to have important economic consequences, which will be investigated, in the remaining sections of this paper.

5 APPLICATION TO OPTIMAL PAVEMENT DESIGN

The simplified economic model used in this study is now described. This model is identical to that proposed by Small and Winston (1988). Consider a section of highway of width W (units of number of lanes) that incurs an annual traffic load of Q (units of equivalent single-axle loads, ESAL), which is assumed to be constant over the life of the pavement. In the case of flexible pavements, the structural number of the pavement, D , is defined as the weighted average of the layers' structural numbers. Assuming that traffic loading is the dominant cause of pavement deterioration, the effects of aging, weathering, and environmental factors are not considered in the model. We use $\rho(D)$ to denote

the number of ESALs that causes the pavement to deteriorate from an initial serviceability p_0 , defined in units of PSI, to a terminal serviceability p_f , for which resurfacing is required, as function of structural number D . Since the outer lane usually incurs the most loading and it is the first to require resurfacing, the time between resurfacing events, $T(\rho)$, is given by

$$T(\rho) = \frac{\rho(D)}{\gamma Q} \quad (14)$$

where γ is the fraction of annual loading Q that occurs in the outer lane. The resurfacing life $T(\rho)$ is assumed to be constant for a given structural number, which is a reasonable assumption under steady-state conditions.

A simple cost model is developed next. Through this model, which consists of capital and maintenance costs for an infinite horizon, optimal values of design structural number that give minimum discounted total pavement cost are determined. As one might expect, both capital and maintenance costs are functions of design structural number, and there is a tradeoff between initial capital and future maintenance.

The capital cost per mile of constructing a new highway, $K(W, D)$, increases linearly with both highway width, W , and design structural number, D . It consists of a constant term, a term for the cost of grading and preparing the roadbed, and a term for the cost of materials. Therefore, the capital cost per mile is expressed as:

$$K(W, D) = k_0 + k_1 W + k_2 WD \quad (15)$$

The cost of maintenance and rehabilitation per mile of pavement, $M(Q, W, D)$, consists of resurfacing cost and routine maintenance cost. We will include only the cost of resurfacing every $T(\rho)$ years, since it is the dominant component. The present value of all future resurfacing costs is given by:

$$M(Q, W, D) = C(W)/(e^{rT(\rho)} - 1) \quad (16)$$

which is the present value of an infinite sequence of payments $C(W)$, made every $T(\rho)$ years. Recall that $T(\rho)$ is related to design structural number D through equation (14).

The total pavement cost is the sum of $K(W, D)$ and $M(Q, W, D)$:

$$TC = [k_0 + k_1 W + k_2 WD] + C(W)/(e^{rT(\rho)} - 1) \quad (17)$$

Dropping terms that do not include D and dividing by W yields the objective function we wish to minimize, the discounted lifecycle cost per lane-mile:

$$TPC = k_2 D + [C(W)/W]/(e^{rT(\rho)} - 1) \quad (18)$$

Table 2
Coefficient estimates for various pavement deterioration models

| | AASHTO | S&W | M&P |
|-----------------------|--------|-------|-------|
| Constant | 13.65 | 12.06 | 12.15 |
| A1 | 9.36 | 7.76 | 6.68 |
| A2 | 4.79 | 3.65 | 3.03 |
| A3 | 4.33 | 3.24 | 2.62 |
| Prediction std. error | 0.67 | 0.63 | 0.42 |

where $C(W)/W$ is the cost of resurfacing one lane-mile of pavement. Though the cost model is fully defined at this point, the relationship between pavement life and design structural number, $\rho(D)$, must be determined through a deterioration model before the objective function can be optimized. In this paper, three different models were used to perform this task. The first model is the original AASHTO model described earlier in this paper. The second deterioration model estimated with the AASHTO road test data has properly accounted for the censored data (Small and Winston, 1988). The authors used the same functional form specified by AASHTO, as given in Equation (10). However, they statistically treated the censored data by using censored regression, also known as Tobit (Greene, 1993), to estimate the coefficients of the model. The third model is the one described earlier in this paper. The parameter estimates of the three models are presented in Table 2. The column titled AASHTO refers to the original AASHTO model. The column titled S&W refers to the Tobit model of Small and Winston. Finally, the column titled M&P (Madanat and Prozzi) refers to the model developed earlier in this paper.

6 EFFECT OF DETERIORATION MODEL ACCURACY ON OPTIMAL DESIGN

The three models were used as input to the objective function in Equation (18) to solve for the optimal design of flexible pavements. As discussed earlier, the objective of flexible pavement design is to determine the optimal structural number for a pavement. For various levels of annual traffic loading, the discounted total pavement costs per lane-mile (TPC) were plotted and the optimal design D^* that minimized TPC were determined. The various economic parameters used in the lifecycle cost minimization, such as the interest rate and the unit costs of pavement materials, are taken from the work of Small, Winston, and Evans (1989) and shown in Table 3. Figures 1a to 1c show the TPC plots predicted by each pavement deterioration model for five different annual traffic loading values. The general shape of the curves are similar for the three models, with a steep increase in TPC to the left of D^* and

Table 3
Economic parameters

| | |
|--|-----------------------------------|
| Interest rate, r | 0.09 |
| Unit cost of flexible pavement materials | \$24,820 per lane-mile-unit of SN |
| Resurfacing cost, $C(W)/W$ | \$113,400 per lane-mile |

a milder increase in TPC to the right. This is a direct result of the higher (discounted) cost of an infinite number of resurfacing events relative to the initial capital costs. At lower structural number, more frequent resurfacing is required to accommodate a given level of traffic loading, thereby increasing TPC exponentially. At higher structural number, the added structural number reduces the frequency of resurfacing and, therefore, makes the discounted cost of resurfacing negligible. In this region, TPC increases nearly linearly at a rate approximately equal to k_2 , the capital cost per lane-mile per unit of pavement structural number.

Optimal structural numbers (SNs) are compared in Figure 2 for the three deterioration models for an annual traffic loading of one million ESAL. When the AASHTO model is used in an economic optimization framework, it yields an optimal SN of 5.5 units, far lower than the designs obtained by using the other two models, which are both around 6.5 units. On the other hand, in current practice, where AASHTO design equations are not based on an economic optimization, the recommended SN is approximately 6.2 units. These discrepancies reveal that while the AASHTO model is statistically biased, and therefore systematically over-predicts pavement life, this bias is somewhat mitigated by an inherent factor of safety that is built in the AASHTO design standards. However, despite this factor of safety, the recommended pavement design in the AASHTO standards is still lower than the cost-minimizing structural number.

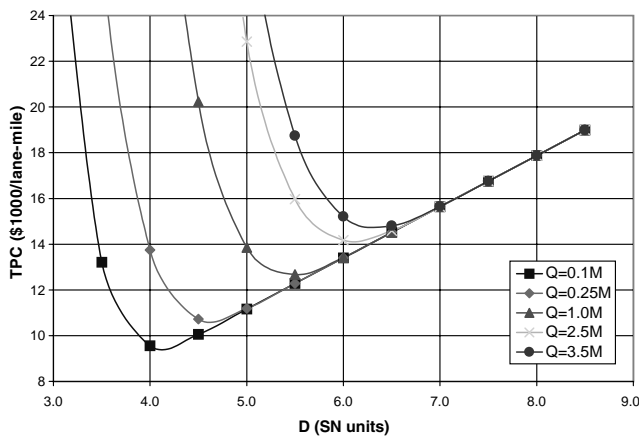


Fig. 1a. Predicted TPC versus D (AASHTO).

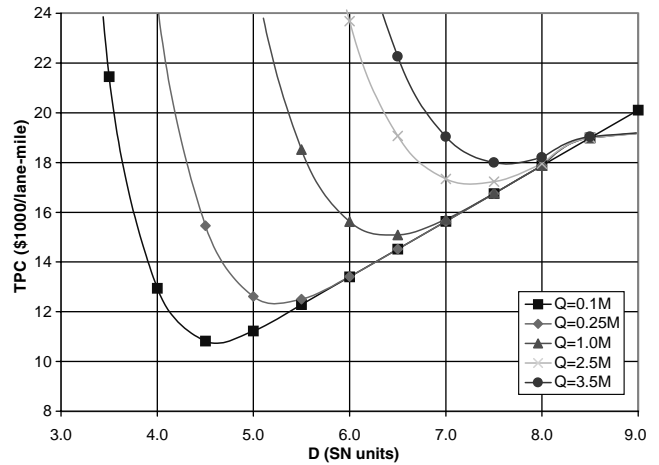


Fig. 1b. Predicted TPC versus D (S&W).

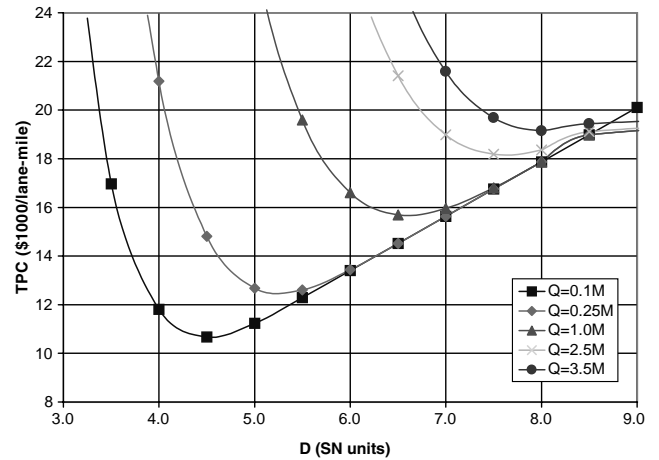


Fig. 1c. Predicted TPC versus D (M&P).

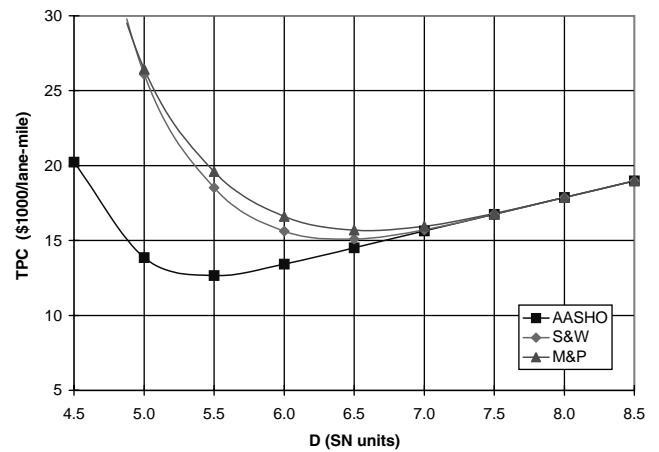


Fig. 2. Comparison of predicted TPC across models for $Q=1.0M$.

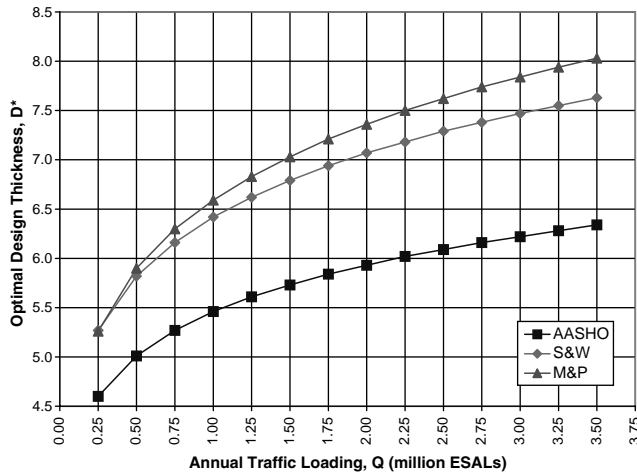


Fig. 3. Optimal design thickness versus traffic loading.

In Figure 3, the optimal structural number is plotted against annual traffic loading. For the three models, optimal design structural number increases with traffic loading, as one would expect, but at a decreasing rate. The results obtained with the second (Small and Winston) model and those obtained with the third model, developed in this paper, are very close to each other because they appropriately account for the censored data. However, the AASHO model suggests much lower structural number to minimize total pavement costs, especially at higher traffic loads.

The sensitivity of the results to the various economic parameters was tested. We varied the interest rate, the unit cost of pavement materials, and the cost of resurfacing. The optimal structural number showed minimal fluctuation, and it was found to be rather insensitive to these parameters. However, Small and Winston (1988) found that the results for flexible pavements are sensitive for different specifications of Equation (10) and concluded that their results are only preliminary. Further investigations are needed to fully explore the sensitivity of the optimal design solution to the deterioration model specification.

7 EFFECT OF DETERIORATION MODEL ACCURACY ON LIFECYCLE COSTS

The analysis described above confirms that the pavement structural numbers obtained by using the AASHO deterioration model within an economic optimization framework are lower than the optimal designs. This is the result of using a biased pavement deterioration model, which over-predicts pavement life. Thus, the frequency of costly resurfacing is underestimated, thereby underestimating total pavement cost. It is important to interpret the graphs shown in Figure 2 correctly: these represent the minimum lifecycle costs of using a particular structural number, as predicted

by the three models. The lifecycle costs actually incurred by a highway agency cannot be predicted exactly, but they are best approximated by the lifecycle costs predicted by using the most accurate deterioration model. Given that its standard error of prediction is the lowest among the three models that use the AASHO road test data, it appears that the model developed in this paper is the most accurate model reviewed. Therefore, using the optimal SN recommended by using the AASHO model (5.5 units) would actually lead to a discounted lifecycle cost of approximately \$20,000 per lane mile (as predicted by the curve representing our model). This is significantly higher than the discounted lifecycle cost of \$15,670 per lane mile obtained with the optimal SN (6.6 units). However, as mentioned earlier, the AASHO design procedures are not based on an economic framework, but rather use the deterioration model together with factors of safety to determine the pavement structural number. Therefore, it is more relevant to compare the optimal pavement designs produced by using the most accurate prediction model to those obtained by following the AASHO pavement design procedure. For an annual traffic loading of one million ESAL, AASHO recommends a flexible pavement structural number of 6.2 units. For this structural number, our model yields a discounted lifecycle pavement cost of \$16,050 per lane-mile (see Figure 2) and a resurfacing interval of 19 years. However, the optimal structural number is 6.6 units for a discounted lifecycle pavement cost of \$15,670 per lane-mile and a resurfacing interval of 27 years, yielding \$380 in savings per lane-mile.

8 CONCLUSIONS

In the first part of this paper, an analysis of the data collected during the AASHO road test was conducted. This analysis is based on the use of probabilistic duration modeling techniques. Duration techniques enable the stochastic nature of pavement failure time to be evaluated as well as censored data to be incorporated in the statistical estimation of the model parameters. Due to the nature of the road failure phenomenon, the presence of censored data is almost unavoidable and not accounting for such data would produce biased model parameters.

In the second part of this paper, we have used a simple economic optimization approach for optimal flexible pavement design. We have shown, through parametric studies, that the deterioration model used as an input to the lifecycle cost minimization has a substantial effect on the results of the design problem. Clearly, a biased pavement deterioration model will lead to suboptimal designs.

In arriving to the above conclusions, we have made a number of assumptions, in order to keep the analysis simple. Among the most important simplifications is that we ignored the effect of routine maintenance, thus probably

underestimating pavement life and overestimating the frequency of resurfacing that is required for a given pavement design. Another significant assumption is that we used pavement deterioration models that were developed using accelerated pavement testing studies (the AASHO road test in this case) to predict pavement life. Models developed from such data generally overestimate pavement life, as they do not account for the effect of materials aging and environmental factors. It is therefore likely that the above two assumptions will tend to mitigate each other. In any case, these assumptions affect the predictions of all three models analyzed. Therefore, while the actual design structural number and costs presented in this paper will change if the above assumptions were relaxed, we expect our general results to hold.

The analysis performed in this paper was deterministic, in that it was based on the lifecycle costs associated with the mean predicted pavement deterioration. Given the non-linearity of pavement maintenance and user costs, there is no guarantee that these costs are in fact equal to the expected lifecycle costs taken over the entire distribution of pavement deterioration and maintenance trajectories.

REFERENCES

- AASHTO, (1986), Guide for design of pavement structures, American Association of State Highway and Transportation Officials, Washington, D.C.
- Garcia Diaz, & A. Riggins, M. (1984), Serviceability and distress methodology for predicting pavement performance, *Transportation Research Record 997*, TRB, National Research Council, Washington, D.C., 56–61.
- Greene, W. (1993), *Econometric Analysis*, MacMillan, New York, NY.
- Highway Research Board (HRB) (1960), The AASHO Road Test, Report 5: Pavement Research, Special Report 61E, National Academy of Science, National Research Council, Washington, D.C.
- Kalbfleisch, J. D. & Prentice, R. L. (1980), *The Statistical Analysis of Failure-Time Data*, John Wiley and Sons, New York.
- Madanat, S. & Mishalani, R. (1998), Selectivity bias in modeling highway pavement maintenance activity effectiveness, *ASCE Journal of Infrastructure Systems*, 4(3), 134–137.
- Paterson, W. D. O. & Chesher, A. D. (1986), On predicting pavement surface distress with empirical models of failure times, *Transportation Research Record 1095*, TRB, National Research Council, Washington, D.C., 45–56.
- Paterson, W. D. O. (1987), *Road Deterioration and Maintenance Effects: Models for Planning and Management*, The Johns Hopkins University Press, Baltimore, MD.
- Paterson, W. D. O., Fossberg, P. E. & Watanatada, T. (1989), World Bank's Highway Design and Maintenance Standards Models (HDM-III): a synthesis of road deterioration and user cost relationships, *Proceedings of the Fifth World Conference on Transport Research*, July 10–14, Yokohama, Japan.
- Prozzi, J. A. & Madanat, S. (2000), Analysis of experimental pavement failure data using probabilistic duration models, *Transportation Research Record 1699*, TRB, National Research Council, Washington, D.C.
- Rauhut, J. B., Lytton, R. L., Jordhal, P. R. & Kenis, W. J. (1983), Damage functions for rutting, fatigue cracking and loss of serviceability in flexible pavements, *Transportation Research Record 943*, TRB, National Research Council, Washington, D.C., 1–9.
- Sebaaly, P. E., Lani, S. & Hand, A. (1995), Performance models for flexible pavement maintenance treatments, *Transportation Research Record 1508*, TRB, National Research Council, Washington, D.C., 9–21.
- Small, K. & Winston, C. (1988), Optimal highway durability, *Academic Economic Review*, 78(3), 560–9.
- Small, K., Winston, C. & Evans, C. (1989), *Road Work*, The Brookings Institution, Washington D.C.