Serviceability and Distress Methodology for Predicting Pavement Performance

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ABSTRACT

In this paper the fundamental aspects in the development and application of a methodology for predicting pavement performance are summarized in terms of three indices: (a) present serviceability index, (b) distress area index, and (c) distress severity index. A statistical procedure used for estimating the parameters of the performance relationships guarantees that the goodness-of-fit between predicted and observed data is maximized. The most salient feature of the proposed methodology is the use of an S-shaped curve that recognizes a change in the rate of deterioration of a pavement as the traffic level accumulates until rehabilitation is needed. Serviceability ratings, based on data obtained from 164 pavement test sections, are used to predict the performance of black-base, hot-mix, and overlay pavements. In addition, the proposed method can be used when pavement performance is ascertained in terms of area and severity distress ratings for several types of pavement distress such as rutting, flushing, raveling, alligator cracking, transverse cracking, longitudinal cracking, and patching.

The purpose of this paper is to summarize recent developments and actual applications of a pavement performance equation that predicts the loss of serviceability or deterioration caused by various types of distress. The proposed model represents an improvement over the original AASHO Road Test performance equation in that it predicts more realistic long-term behavior. This is achieved through the use of a sigmoidal or S-shaped curve that recognizes the ability of a pavement to reduce its rate of deterioration as the traffic level approaches the end of the service life of the pavement. This behavior, for example, is typical of pavements that have received adequate routine maintenance in the past. To evaluate the parameters in the performance model, a least-squares curve fit technique is employed using field measurements from the data base for flexible pavements available at the Texas Transportation Institute (1). The types of pavements considered along with the number of test sections evaluated are as follows: black base, 51 sections; hot-mix asphaltic concrete, 36 sections; and overlays, 77 sections.

The data for each test section consisted of values of the present serviceability index as a function of the number of 18-kip equivalent axle loads. In addition, the structural performance of the pavement was evaluated in terms of distress severity and area for the following distress types (in each case the primary variable correlated with the distress type is shown in parentheses): rutting (N-18), alligator cracking (N-18), patching (N-18), flushing (ADT), raveling (ADT), longitudinal cracks (time), and transverse cracks (time). For the primary variables, N-18 and ADT represent the number of 18-kip single-axle loads and the average daily traffic, respectively; in addition, time represents the number of months since initial construction.

The paper is divided as follows. First background information that pertains to the development of the AASHO highway performance equation is presented. Second, the development and characteristics of the proposed sigmoidal or S-shaped curve are described. Third, the procedure for determining the design constants for the curve, using present serviceability index data, is presented. Fourth, the prediction of pavement distress using the proposed methodology is discussed. Finally, an actual application of the method to predict the functional and structural performance of Texas pavements is presented.

GENERAL BACKGROUND ON PERFORMANCE EQUATIONS

Types of Performance

In the 20 years since the AASHO Road Test began, the idea of performance has been accepted and broadened to accord with the measures of service that the pavement provides. Because of this, it is now possible to define roughly three types of performance: functional, structural, and survival (2).

1. Functional performance: This is the measure that was adopted by the AASHO Road Test; that is, the present serviceability index, which measures the quality of riding conditions from the point of view of the traveling public.
2. Structural performance: The deterioration of structural performance is measured by the appearance of various forms of distress and their relative importance in triggering decisions to maintain or to rehabilitate a pavement. These measures include roughness, cracking (several types), rutting, flushing, raveling, failures (potholes), and patches in flexible pavements. The measures for rigid pavements include spalling, cracking, and joint problems such as pumping, failures, and faulting. Because structural performance is visible or measurable, whereas functional performance is primarily subjective, there have been several attempts to relate the two.
3. Survival: The survival of a pavement is determined by the amount of time that it lasts before major maintenance or rehabilitation must be performed. Survival is measured by the probability that a given pavement is still in service a number of years after its construction. Historical records may be used to develop such survivor curves, which are important in projecting budget levels for maintenance and rehabilitation work.

Each of these kinds of performance has its own use in serving the public. The latter two are of principal importance to the agency that is responsible for keeping a roadway network in good operating condition.

The form of the AASHO equation is

$$g = (W/\rho)^{\beta} \quad (1)$$

where

g = damage function, which is a normalized variable that ranges from 0 to 1 as distress increases or as functional performance or survival probability decreases;
W = quantity of normalized load or climatic cycles, or the total elapsed time to reach a given level of g;
ρ = quantity of normalized load or climatic cycles, or the total elapsed time until g reaches a value of 1 (it is usually assumed to be a function of the structural variables); and
β = a power that dictates the degree of curvature of the curve relating g to the ratio of W/ρ; a high value of β (greater than 1) indicates that g remains low over the majority of the life of the pavement, whereas a low value of β (less than 1) indicates a high value of g over the life of the pavement.

The damage function in the AASHO design equation is defined as a serviceability index ratio:

$$g = (P_o - P)/(P_o - P_f) \quad (2)$$

where P_o is the initial serviceability index, and P_f is the minimum serviceability index (in the AASHO design equation this value is equal to 1.5). Combining Equations 1 and 2, the AASHO design equation can be rewritten as

$$P = P_o - (P_o - P_f)(W/\rho)^{\beta} \quad (3)$$

A graphical representation of Equation 3 is shown in Figure 1.

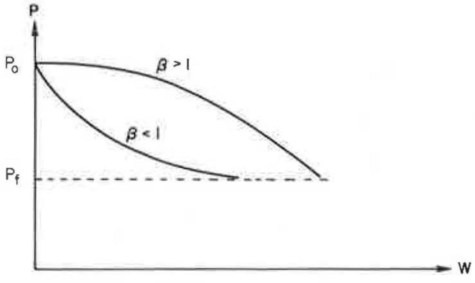


FIGURE 1 AASHO performance curve.

Alternative Forms of Functional Performance Equations

The shape that a functional performance curve should take can be deduced from the boundary conditions placed on the serviceability index scale as well as from long-term observations of field data. The serviceability rating scale ranges between 0 and 5 and, as it is defined, can be neither greater than 5 nor less than 0. As pavement roughness increases, the serviceability rating will decrease and will approach, but not drop below, a serviceability rating of 0 no matter how much traffic passes over the pavement. Thus the performance curve starts out horizontally bounded from above by a rating of 5. As load repetitions increase, the curve is bounded from below by a rating of 0, a value that it approaches as an asymptote. These boundary conditions imply that a functional performance curve should be S-shaped.

The form of the AASHO design equation (Equation 1) assumed that the serviceability-index-versus-traffic curve never reverses its curvature, as shown in Figure 1. By way of contrast to this assumed form of equation, a number of observed serviceability-index-versus-traffic relations have shown a reversal of curvature (see Figure 2).

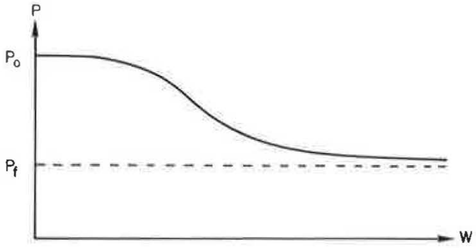


FIGURE 2 S-shaped performance curve.

The S-shaped feature of the curve shown in Figure 2 requires an equation of the form

$$(P_o - P)/(P_o - P_f) = \exp[-(\rho/W)^{\beta}] \quad (4)$$

which can be rewritten as

$$P = P_o - (P_o - P_f)\exp[-(\rho/W)^{\beta}] \quad (5)$$

In this paper the S-shaped performance function defined in Equation 5 has been considered. Obviously, there are many choices for this function; the following list of considerations is helpful in deciding what particular choice to use.

1. The function must have a maximum (minimum) value at traffic level (or time) equal to zero and must be strictly decreasing (increasing) as the traffic level increases.
2. The function cannot have negative values; indeed, if the performance value is standardized to be between 0 and 1, the particular choice of the function cannot have values outside this range as traffic or time increases.
3. The function must have at least one parameter so that a family of pavements may be represented for different values of the parameter or combinations of parameter values in the case of several parameters.
4. The structure of the performance function must be suitable for an efficient estimation procedure of the parameters on the basis of observed data.

It is easy to verify that all of these conditions are satisfied by Equation 5. This equation has been investigated and validated by using an extensive data base for flexible pavements available at the Texas Transportation Institute. Previous studies have also demonstrated the validity of Equation 5 in predicting pavement performance (1,3-6).

PROCEDURE FOR DETERMINING DESIGN CONSTANTS

Assuming that P_o is known, the purpose of this section is to develop a statistical procedure to determine the constants P_f, ρ, and β on the basis of observed performance data for a given type of pavement.

The performance relationship (Equation 5) can be expressed as

$$P_o - P = \alpha \exp[-(\rho/W)^{\beta}] \quad (6)$$

where

$$\alpha = P_o - P_f \qquad (6a)$$

Taking the natural logarithm of Equation 6 yields

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - (\rho/W)^{\beta} \qquad (7)$$

which can also be written as

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - \rho^{\beta}(1/W)^{\beta} \qquad (8)$$

Using the transformation $e^{\tau} = 1/W$, Equation 8 becomes

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - \rho^{\beta}(e^{\beta})^{\tau} \qquad (9)$$

which is equivalent to

$$z = a - bc^{\tau} \qquad (10a)$$

where the variables of substitution are

$$z = \mathrm{Ln}(P_o - P) \qquad (10b)$$

$$a = \mathrm{Ln}(\alpha) \qquad (10c)$$

$$b = \rho^{\beta} \qquad (10d)$$

$$c = e^{\beta} \qquad (10e)$$

Given a collection of m data points (P_i, W_i), where P_i is the serviceability index corresponding to a traffic level W_i, and i = 1, 2, ..., m, the remaining portion of this section deals with the development of a statistical procedure to find a, b, and c on the basis of observed data.

Specifically, the data can be computed as follows:

1. Find $z_i = \mathrm{Ln}(P_o - P_i)$ for i = 1, 2, ..., m, and
2. Find $\tau_i = \mathrm{Ln}(1/W_i)$ for i = 1, 2, ..., m.

Therefore, the observed values of P_i and W_i are transformed into values of z_i and τ_i, respectively. The statistical model to be used is defined as

$$z_i = a - bc^{\tau_i} + \epsilon_i \qquad (11)$$

where ϵ_i is the random error corresponding to the value z_i associated with τ_i.

The basic procedure to estimate the parameters a, b, and c is the well-known least-squares method. This method computes a, b, and c in such a way that the quantity $\sum_{i=1}^{m} \epsilon_i^2$ is minimized. This quantity can be obtained from Equation 11 as

$$\sum_{i=1}^{m} \epsilon_i^2 = \sum_{i=1}^{m} (z_i - a + bc^{\tau_i})^2 \qquad (12)$$

The necessary (and in this case sufficient) conditions for a minimum are given by

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial a = 0 \qquad (13a)$$

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial b = 0 \qquad (13b)$$

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial c = 0 \qquad (13c)$$

These conditions can be shown to be equivalent to

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i}) = 0 \qquad (14)$$

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i})c^{\tau_i} = 0 \qquad (15)$$

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i})\tau_i c^{\tau_i - 1} = 0 \qquad (16)$$

It is noted that Equations 14 and 15 are linear in a and b; therefore both parameters can be obtained in terms of z_i, τ_i, and c. The corresponding results are as follows:

$$a = \left[\left(\sum_{i=1}^{m} C^{2\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right] \qquad (17)$$

$$b = \left[-m \cdot \left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right) + \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_j\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{j=1}^{m} C^{\tau_i}\right)\right] \qquad (18)$$

The values of a and b given by Equations 17 and 18 can be substituted into Equation 16 to obtain the following final result:

$$\begin{aligned}\sum_{i=1}^{m} Z_j \tau_i C^{\tau_i} - \Bigg\{&\left[\left(\sum_{i=1}^{m} C^{2\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right)\right] \\ &\div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right]\Bigg\} \cdot \left(\sum_{i=1}^{m} \tau_i C^{\tau_i}\right) \\ &+ \Bigg\{\left[-m \cdot \left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right) + \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_j\right)\right] \\ &\div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{j=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right]\Bigg\} \\ &x\left(\sum_{i=1}^{m} \tau_i C^{2\tau_i}\right) = 0\end{aligned} \qquad (19)$$

Equation 19 can be solved for c by using a trial-and-error method or a simple numerical analysis method. Once c is determined, a and b can be computed from Equations 17 and 18, respectively, and their corresponding values can be used to estimate α (and thus P_f), ρ, and β from Equations 10c, 10d, and 10e.

In the case of the method discussed in this paper, the observed data P_i and W_i are transformed into z_i and τ_i, respectively. It is noted that both parameters a and b are linear functions of the transformed data for a fixed choice of c. Therefore, a valid strategy is to consider a collection of c values and for each one conduct a regression analysis, while monitoring the acceptability of the c value in terms of Equation 16 or its equivalent Equation 19. The error of the estimation procedure is summarized by the degree at which Equation 19 is held as an equality. Of course, for each choice of c the variance of the estimates of the parameters a and b can be measured by using well-known regression analysis results. These results are not given in this paper but are available in any textbook on elementary statistics that includes a discussion of the variance of the estimates in regression analysis.

PREDICTION OF PAVEMENT DISTRESS

Pavement distress is best represented in two separate components: density and severity. Density may be expressed as either the percentage of the total pavement surface area that is covered by the distress, or total crack length per unit area or crack spacing or similar measures. Severity may be expressed as either an objective or subjective measure. Examples of objective measures are crack width, crack depth, and relative displacement at a joint. Subjective measures may be assessed reliably by comparing the observed distress with photographs of different levels of severity. The severity may be described as none, slight, moderate, or severe and may be given numerical ratings such as 0, 1, 2, and 3, respectively, or be assigned numbers that are proportional to these in a range between 0 and 1. The change of either area or severity of distress can be evaluated by using the previously discussed equations.

To study the behavior of the area covered by a given type of distress and the corresponding level of severity, two indices are introduced: (a) the distress area index, and (b) the distress severity index. Each of these indices represents a number between 1 and 0 that decreases as the level of traffic is increased. Note that the present serviceability index has a similar behavior, with the exception that it decreases from P_o to P_f.

Specifically, the distress area index decreases from a value A_o ($A_o \leq 1$) to a value A_f ($0 \leq A_f \leq A_o$) as the traffic increases; similarly, the distress severity index decreases from a value of S_o ($S_o \leq 1$) to a value S_f ($0 \leq S_f \leq S_o$) as the traffic level increases. Note that both the area and severity indices are reduced as traffic increases; that is, a recently rehabilitated pavement will have indices close to one, as opposed to pavements in need of rehabilitation, which will have indices close to zero.

The distress area index (A) is expressed by a relationship similar to that of Equation 3, namely,

$$A = A_o - (A_o - A_f)\exp[-(\rho/W)^{\beta}] \qquad (20)$$

Similarly, the distress severity index (S) is expressed as

$$S = S_o - (S_o - S_f)\exp[-(\rho/W)^{\beta}] \qquad (21)$$

Using the A, S, and W data from the Texas Transportation Institute data base it is possible to estimate A_f, S_f, ρ, and β following the procedure described by Equations 5-19 for each of the following types of distress: rutting, raveling, flushing, alligator cracking, longitudinal cracking, transverse cracking, and patching.

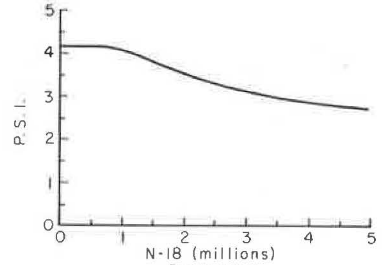
APPLICATION, SUMMARY, AND CONCLUSIONS

The S-shaped performance curve is found to adequately describe the performance of flexible pavements in Texas as a result of increased traffic levels. This behavior has been analyzed primarily in terms of the decrease in the present serviceability index (PSI) as a function of the number of 18-kip equivalent axle loads. The proposed performance curve was developed on the basis of observed data for pavements in each of the following categories: black base, hot mix, and overlays. A more detailed description of the curve fit parameters, along with the original data, can be found in the report by Garcia-Diaz et al. (1). The data in Table 1 give the number of test sections in each category along with mean value and minimum and maximum observed values of the design parameters. The mean values of the curve fit parameters were obtained from the statistical procedure described earlier in this paper. Figure 3 shows the average performance curves obtained by using these design parameters (see Table 1) for each of the three pavement types.

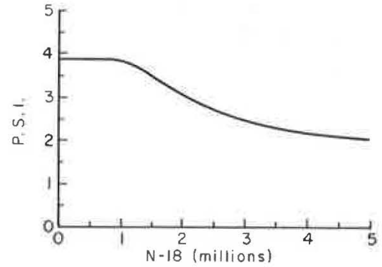
The analysis of the data revealed four possible cases for the curve fit. Typical test sections for

TABLE 1 Serviceability Performance Curve Parameters by Pavement Type

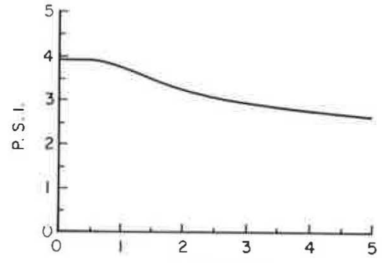
Pavement Type	Black Base	Hot Mix Asphalt Concrete	Overlays
Number of Test Sections	51	36	77
ρ (mean)	2.321	1.960	1.974
ρ (min)	0.005	0.100	0.013
ρ (max)	17.239	11.098	9.188
β (mean)	1.337	1.952	1.196
β (min)	0.300	0.095	0.095
β (max)	6.277	7.259	2.893
P_o (mean)	4.15	3.87	3.92
P_o (min)	2.79	2.86	2.07
P_o (max)	4.77	4.78	4.88
P_f (mean)	1.962	1.661	2.121
P_f (min)	0.000	0.000	0.004
P_f (max)	4.295	4.305	4.391



Black Base

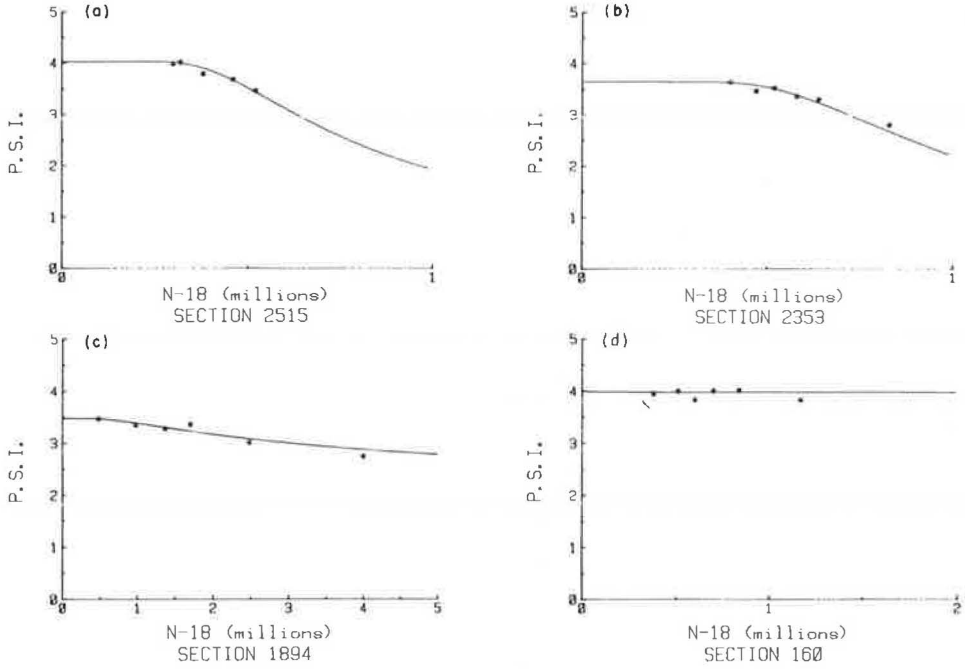


Hot Mix



Overlays

FIGURE 3 Performance curves from the mean design parameters.



SECTION 2515

SECTION 2353

SECTION 1894

SECTION 160

FIGURE 4 Typical sections for the four cases of serviceability performance.

each case are shown in Figure 4. A description for each case follows.

1. Case 1 (Figure 4a): $\rho > 1$, $\beta > 1$, and $P_o > P_f$. Note that the complete S-shaped pattern can be distinguished. The percentage of pavements of this type = 26.9. The example shown in for a black base (test section 2515).
2. Case 2 (Figure 4b): $\rho > 1$, $\beta > 1$, and $P_o > P_f$. Note that the upper half of the S-shaped curve is observed. The percentage of pavements of this type = 28.6. The example shown is for an overlay (test section 2353).
3. Case 3 (Figure 4c): $\rho > 0$, $\beta < 1$, and $P_o > P_f$. Note that the lower half of the S-shaped curve is observed. The percentage of pavements of this type = 21.3. The example shown is for an overlay (test section 1894).
4. Case 4 (Figure 4d): $\rho > 0$, $\beta = 0$, and $P_o = P_f$. Note that no noticeable curve is observed. The percentage of pavements of this type = 21.3. The example shown is for an overlay (test section 160).

TABLE 2 Primary Distress Type and Curve Fit Parameters by Pavement Type

Pavement Type	Black Base	Hot Mix Asphalt Concrete	Overlays
Type of Distress	Alligator Cracking Severity	Alligator Cracking Area	Transverse Cracking Severity[a]
ρ(mean)	1.19	0.93	85.57
ρ(min)	0.14	0.07	24.13
ρ(max)	3.01	3.63	194.83
β(mean)	2.54	3.43	1.47
β(min)	0.89	0.50	0.50
β(max)	8.78	18.21	5.52

[a] The ρ and β terms for this case are determined in terms of the number of months the pavement has been in service.

The S-shaped performance curve is also applicable in the analysis of distress data. For this case the assumptions that $A_o = S_o = 1$ and $A_f = S_f = 0$ will simplify the analysis because only the parameters ρ and β remain to be estimated. These values can then be used to develop a performance curve for each of the two distress indices (area and severity). The development and application of distress models for rutting, alligator cracking, longitudinal cracking, and transverse cracking are summarized elsewhere (1). Sample results of this analysis for distress types found to be critical [using the method of discriminant analysis as discussed by Allison et al. (3)] are given in Table 2.

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Development and Testing of a Portable Microprocessor-Based Capacitive Weigh-in-Motion System

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ABSTRACT

In this paper current techniques for the measurement of dynamic axle load are reviewed and the development and testing of a prototype portable weigh-in-motion (WIM) system at the University of Nottingham, England, are described. Current techniques for dynamic axle load detection tend to be characterized by a need for permanent sensors within the road pavement that can be both costly to install and maintain. A portable WIM system that uses a temporary capacitive weighmat has been developed at the National Institute for Transport and Road Research of South Africa, and extensive tests have revealed that the system can provide accurate results for accumulated axle loadings over large samples of vehicles, but that results for individual axle loads are subject to larger errors. In an attempt to improve the reliability of weight data from the capacitive sensor, a new microprocessor-based detector unit was developed at the University of Nottingham. Some current WIM systems developed in the United States and in the United Kingdom are evaluated, and the theory of operation of the capacitive sensor is described. Laboratory tests undertaken to determine the response of the sensor to controlled loading are discussed in detail, and the prototype detector unit is described along with the results of field trials conducted on a main U.K. highway. The results of these trials indicated that improvements over previous capacitive WIM systems had been achieved and that the combination of a commercially available capacitive sensor and a new microprocessor-based detector unit provided axle load data that was within ±15 percent of static weights. A commercially available capacitive WIM system, developed by the Golden River Corporation from this prototype system, is described along with a second-generation automatic weighing and classification system that was under development at the time of writing.

The geometric design of a highway is based on estimates of the expected total flow of traffic and the expected changes in the traffic flow over the design life of the road. Pavement design depends on other factors, and structural damage is caused almost exclusively by commercial vehicles (1). The normal practice in Europe and North America is to express the expected number of commercial vehicles as the cumulative total of equivalent standard axles. In the United Kingdom this is calculated by multiplying the expected number of commercial vehicles by two design factors, one characterizing the average number of axles per commercial vehicle and the other the number of standard axles per commercial axle.

The accuracy of the standard axle estimates is dependent on the reliability of data on past and present trends in axle loadings, which are in turn related to the size of the vehicle samples monitored. When manual classification and recording of axle loads are necessary, sample periods may be too short to allow accurate annual estimates of traffic loading to be deduced. In addition, axle load sta-