

Probabilistic Design of Flexible and Rigid Pavements Using AASHTO Equations

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This paper applies statistical and probabilistic methods to the design of pavements, using the existing AASHTO equations. The major purpose for applying such methods was to assess the variation of the pavement geometry as a function of the variabilities of design factors. A linear first-order approximation was applied on the AASHTO pavement design equation to determine the effect of the variation of traffic and soil support value on the variation of structural number for flexible pavement and the effect of the variation of traffic and concrete flexural strength on the variation of slab thickness for rigid pavements. The mean and variance equations for the pavement geometry were derived by using Taylor's series expansion about the mean. Nomographic solutions of these equations are provided, which would prove helpful in practical applications. The final design thickness of the pavement for a specific service life and design parameters variability is obtained by combining the nomographic solutions with the least cost concept, thereby satisfying economy, performance and statistical design requirements. Examples to illustrate the use of this technique in the design of both flexible and rigid pavements are presented.

The design of pavement systems, from a practical point of view, involves the selection and assignment of specific values for several design factors. However, the design input parameters are rarely, if ever, unique or constant values. Strictly speaking, every design factor studied and analyzed possesses some degree of variability and randomness in its measurement. It is not surprising, for instance, in a given design situation, to find subgrade support value for a single soil varying rather considerably over a wide range. The recent recognition of these probabilistic random properties of design and material factors has brought great attention to the use of statistical concepts within the field of pavement technology. Application of statistical and probabilistic methods to the design of pavement systems would seem to be, therefore, an essential step toward improving existing design procedures. Probabilistic techniques have been used extensively for several years in various areas of engineering. The consideration of material variations, traffic load uncertainties, and soil properties variabilities in the design of pavements has been strongly advocated by several researchers. Probabilistic methods have been applied to flexible pavement design and analysis by Darter et al. (1–4) and McManus and Barenberg (5). Applications to rigid pavements were made by Kher and Darter (6, 7).

Even though all pavement-design methods, in particular the AASHTO design, consider the effect of factors such as subgrade, pavement layer strength, traffic characteristics, and

environmental conditions on the pavement geometry (thickness), these methods do not take into account the variabilities of these design parameters. On the basis of an extensive survey, Von Quintus et al. (8) indicated that pavements designed using the conventional AASHTO method did not last the entire intended 20-year period. They observed that in most cases, the in-service life of these pavements was between 8 and 12 years. Furthermore, Von Quintus et al. stated that highways in many of the urban and suburban areas throughout the United States are subjected to unusually heavy traffic volumes and traffic loading uncertainties, which often cause pavements to deteriorate early. Hence, designing pavements using the conventional AASHTO method, which is considered to be a deterministic solution, would prove to be insufficient. However, the inclusion of probabilistic concepts in the AASHTO equations may provide a better and more realistic pavement-design method. The introduction of such a concept, thus, represents an attempt to amplify the AASHTO design methodology.

The objective of this study was to extend on the AASHTO design equations and to show how the uncertainties in design factors affect the variation of thickness requirement in pavements. The AASHTO Interim Guide equations for flexible and rigid pavements provide the basic structural design models. Through a first-order linear approximation of the AASHTO equations, the impact of the most significant design parameter variabilities (such as traffic and soil support value for flexible, and traffic and concrete strength for rigid) on the variation of pavement geometry was evaluated and quantified for three types of pavements—secondary ($P_t = 2.0$), primary ($P_t = 2.5$), and premium ($P_t = 3.0$). In order to satisfy both structural design requirements and economy, the above solution (presented in a nomographic form) was combined with the least-cost analysis concept; thereby, producing a design thickness corresponding to a predetermined structural reliability.

GENERAL FRAMEWORK

A combination of existing design procedure (9), statistical techniques (10–12), and the least-cost concept (3, 13) was used in developing the statistical design thickness requirements for both flexible and rigid pavements. The AASHTO equations form the basis for structural design. For flexible pavements, this equation relates traffic repetitions to structural number, soil support value, and regional factor and is written sym-

bolically as follows:

$$\log W = 9.36 \log(SN + 1) - 0.20 + \log[0.37(4.2 - P_t)] \\ \div \{0.40 + [1,094/(SN + 1)^{5.19}]\} \\ + 0.38(SSV - 3.0) - 0.97 \log(R_f) \quad (1)$$

where

W = total number of 18-kip equivalent axle loads (EAL),
 SN = weighted structural number,
 P_t = terminal serviceability index,
 SSV = soil support value, and
 R_f = regional factor.

On the other hand, for rigid pavements, the AASHTO model expresses traffic repetitions in terms of concrete slab thickness and working strength as well as modulus of subgrade reaction. This equation is presented mathematically as follows:

$$\log W = 7.35 \log(D + 1) - 0.06 + \log[0.333(4.5 - P_t)] \\ \div \{1 + [1.624 \times 10^7 / (D + 1)^{8.46}]\} \\ + (4.22 - 0.32P_t) \left(\log \frac{S_c}{690} \lambda \right) \quad (2)$$

where

D = concrete slab thickness in inches,
 S_c = concrete working strength (psi),
 E = concrete modulus of elasticity (psi),
 k = modulus of subgrade reaction (pci), and
 $\lambda = (D^{0.75} - 1.132) / \{D^{0.75} - [18.42 / (E/k)^{0.25}]\}$.

One tacit drawback, however, in the AASHTO design procedure is that it does not account for the variations in the design factors. To overcome this deficiency, a first-order linear approximation was applied to Equations 1 and 2 to estimate the variation of pavement design thickness as a function of the variabilities of other design factors. The final design process entails the combination of the aforementioned method with the least-cost concept; thereby, satisfying both statistical safety and economy.

VARIABILITIES IN PAVEMENT DESIGN INPUTS

It has been said that nearly every measurable component used in pavement design possesses some degree of variability. Indeed, to those who are closely affiliated with pavement design and performance, the word variability has much meaning. In designing pavement systems, one must inevitably estimate many inputs from information that is, by and large, limited. In many cases, such design inputs vary and are rarely unique or constant. Thus, available deterministic design procedures are, in a sense, inadequate.

However, if input parameters' variabilities are identified and magnitudes quantified, they can be incorporated into the design process to produce a more realistic design procedure. Many of the variabilities have been reported in the literature, namely by Darter et al. (1) and by AASHTO (9). In the current research context, the variabilities (expressed in terms of a mean value and a coefficient of variation) of such design

inputs as traffic and subgrade strength (for flexible) and traffic and concrete flexural strength (for rigid) pavements will be considered, and their impact on thickness design will be evaluated. A brief discussion of the variation of these factors follows.

Variations in Traffic

Perhaps the most variable and uncertain design input is the traffic loadings and traffic repetitions expected over the life of the pavement. Many factors give rise to uncertainty in traffic prediction, which include social and economic factors as well as others. Such variabilities and uncertainties become most evident on high-volume freeways. On the New Jersey Turnpike, pavement design was based on 20 million applications, but estimates indicated that the 20-year count was over 90 million vehicle applications. In Kentucky, using a new method, Deacon and Lynch (14) observed that the equivalent wheel loads over a 20-year period for 20 locations varied dramatically from actual counts. They stated that, in general, actual traffic will usually fall between one-half and two times the best estimate.

Variations in Subgrade Strength

Regardless of the type of pavement and the method of design being used, usually one starts by assessing the soil areas that are expected to be the subgrade. Inescapably, several laboratory and field tests must be conducted. In many cases, one would be surprised to find that even within presumably homogenous soil areas, soil properties exhibit considerable variation. Several researchers have emphasized this fact (10,9,13) and have further observed that variations and dispersions in different soil properties vary widely, as indicated by the coefficient of variation (standard deviation divided by the mean). They also noted that the highest coefficient of variation occurred in the strength properties of soils.

In the AASHTO design procedure for flexible pavements, the strength of the subgrade is expressed by the soil support value, SSV. However, the arbitrary manner in which soil support value was introduced into the AASHTO procedure makes it an input value that cannot be directly obtained by testing and, therefore, must be correlated (in one way or another) to measurable soil strength properties (California bearing ratio, CBR; triaxial strength; resilient modulus, M_R ; etc.) Such correlations were established by Utah (15,16) and Van Til et al. (17). The latter was based on a theoretical layered analysis. With the suggested values in these references of SSV for different CBR and M_R , a regression analysis was performed, which resulted in the following equations:

$$SSV = 1.57 + 1.46 \ln CBR \\ r^2 = 0.989, SE = 0.069 \quad (3)$$

and

$$SSV = -0.032 + 2.73 \ln M_R \\ r^2 = 0.996, SE = 0.103 \quad (4)$$

where CBR is in percent and M_R in units of 10^3 psi.

In order to express mathematically the variation of SSV in terms of the variations of CBR and M_R , a first-order linear approximation for the variance (discussed in detail in the next section) was applied to Equations 3 and 4, and the result is the following:

$$\text{Var}[\text{SSV}] = 1.46\overline{\text{CBR}} \cdot \text{CV}^2[\text{CBR}] \quad (5)$$

and

$$\text{Var}[\text{SSV}] = 2.73\overline{M}_R \cdot \text{CV}^2[M_R] \quad (6)$$

where

$\text{Var}[\text{SSV}]$ = the variance of SSV,
 $\overline{\text{CBR}}$ and \overline{M}_R = the mean of CBR and M_R ,
 respectively, and
 $\text{CV}[\text{CBR}]$ and $\text{CV}[M_R]$ = the coefficient of variation of
 CBR and M_R , respectively.

Figure 1 is a graphical presentation of Equations 3, 4, 5, and 6. It should be pointed out that Equations 3 and 4 are

considered to be the mean value equations for SSV, as defined by the first-order linear approximation.

On the other hand, the strength of soil in the AASHTO design method for rigid pavements is designated by the modulus of subgrade reaction, k , which is determined by means of the plate-bearing test. Usually, this test is time consuming and, except for special cases, is rarely performed for design of rigid pavements. Typical values can be easily obtained for different soils from tables such as the Unified Soil Classification system. These values are not expected to vary greatly and are justified for design of rigid highway pavements (12). Furthermore, it can be seen (from Equation 2) that the modulus k is relatively insensitive in the analysis of rigid pavements. Therefore, the effect of the variation of subgrade modulus on the variation of concrete slab thickness will be excluded from this analysis.

Variation in Pavement Layer Strength

Generally speaking, the variability of layer strength is a function of the type of material, layer location, and (probably

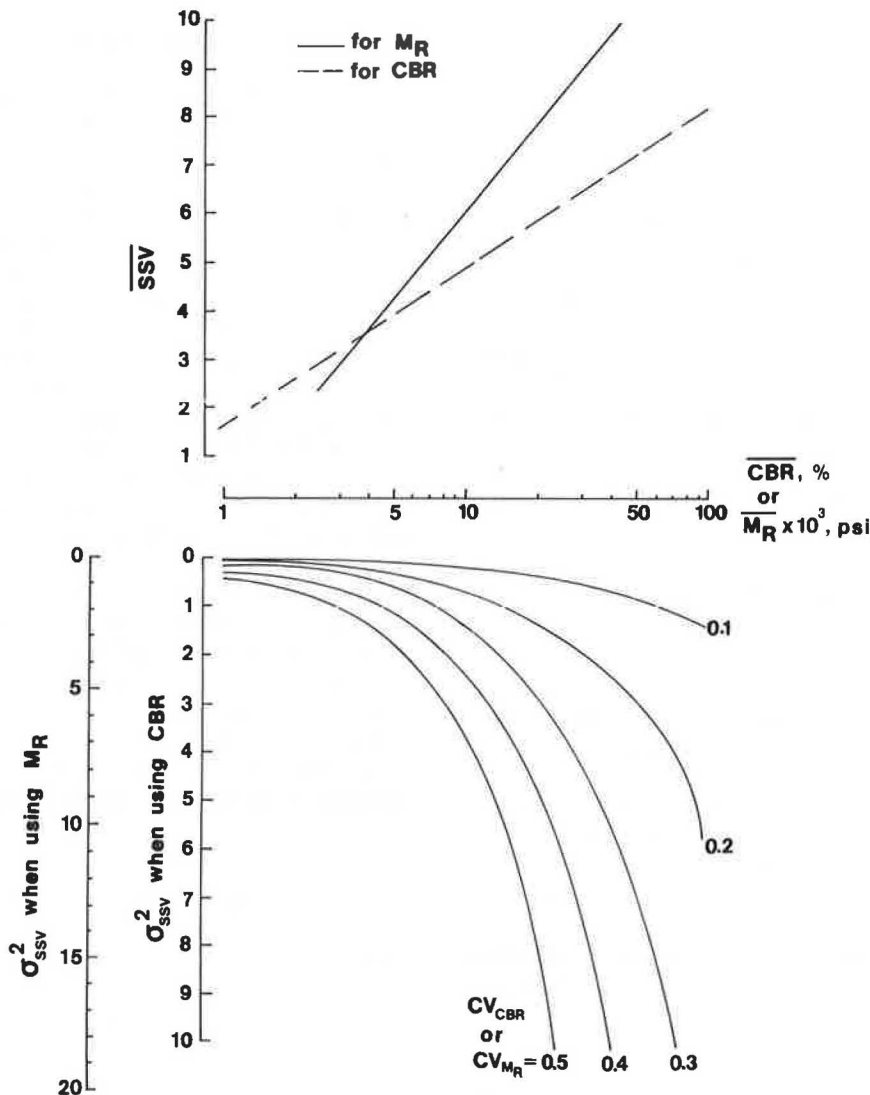


FIGURE 1 Variation of soil support value related to variation of CBR or M_R .

most important) the construction control exercised to obtain uniformity in the material. Strength variability in pavement layer, in general, is expected to decrease as one proceeds toward the natural foundation soil. This may be explained by the fact that, during the life of the pavement, layers (especially those on the surface) are exposed to many performance variations that are directly related to traffic loadings, environmental conditions, maintenance procedures, and occurrence of distresses along the pavement.

In addition, it has been observed (12) that the variability in layer strength data tends to increase as the mean strength increases, especially in flexible pavements. Furthermore, the relationship between the standard deviation and the mean suggests that the use of the coefficient of variation, CV, is a more practical way of describing strength variability.

From the data presented in the literature (12,18), it can be seen that typical values for the CV of the subbase and base range from 15 to 40 percent, whereas the asphalt concrete surface layer has a slightly higher CV. The modulus of rupture variability data of Portland cement concrete, on the other hand, has been found to be a function of the mean compressive strength value (18) with common CV values ranging from 10 to 15 percent.

PROBABILISTIC DESIGN OF PAVEMENTS

Selecting the most effective and economical design for a given project is imperative to the pavement engineer and at the core of all engineering practice. The overwhelming demand for better design processes in pavement technology arises from the limited pavement funds, materials, public need for better performance, and less traffic delay due to maintenance. Therefore, choosing an optimal design is a matter of vital importance. Application of probabilistic techniques to pavement design and analysis may help attain greater optimization in many ways. Probabilistic techniques allow direct consideration of variations and uncertainties in design inputs and, thus, the thickness of the pavement may be increased, depending on these variations, to reduce occurrence of random distresses based on a given reliability. The reliability of the design, R , can be defined as the probability that the actual thickness will not exceed the design thickness obtained,

$$R = P(t_a \leq t) \quad (7)$$

where

- P = probability of occurrence,
- t_a = actual thickness for which the pavement should have been designed given certain conditions, and
- t = thickness for which pavement was designed given the same certain conditions as t_a .

Alternatively, the probability of failure, P_f , can be defined as

$$P_f = 1 - R \quad (7a)$$

To evaluate the reliability of a design, one must first assess the variation of pavement thickness based on the variations of the design inputs; i.e., estimate the mean and variance (or standard deviation). The AASHTO design equations (Equa-

tions 1 and 2) are used for this purpose. The two basic assumptions made in this analysis are (a) the design input parameters are independent random variables, and (b) the pavement thickness is normally distributed.

The earlier assumption could be easily verified, whereas the latter may have one drawback in that the tails of a normal distribution tend toward infinity and can take on negative values, where, in reality, pavement thickness is positive and bounded. However, since the reliability, R , is comparatively larger for pavements, the effect of the exact shape of the tails is relatively insignificant, and, thus, the assumption of normality seems viable. Furthermore, because, in practice, the choice of the probability distribution may also be dictated by mathematical convenience, and because the normal distribution is mathematically simple with a wide availability of information (probability tables) associated with it, the normal model is frequently used in pavement engineering—even when, at times, there is no basis for such a model.

Since t_a is normal, it should be transformed to standard normal (i.e., normal variate with mean zero and standard deviation of one) so that R can be estimated from normal tables.

$$z_t = \frac{t - \bar{t}}{\sigma_t} \quad (8)$$

where

- z_t = standard normal variate of t ,
- \bar{t} = mean thickness of pavement, and
- σ_t = standard deviation of pavement thickness.

The reliability may be found from normal tables as

$$R = P(z_{t_a} \leq z_t) \quad (9)$$

In other words, the reliability is the area under the standard normal, bounded between $-\infty$ and z_t . Determination of \bar{t} and σ_t is the next task.

Mean and Standard Deviation of Pavement Thickness

To assign reasonable values for the pavement thickness, whether flexible or rigid, it would be necessary to relate it to some input design parameters. Such relationships, as mentioned earlier, are readily available in the literature—one of which is the AASHTO design equations. However, these best-fit formulas between the pavement thickness, t (dependent variable), and several design inputs, X_i (independent variable), might qualify as the mean-value function of the dependent variable, in the light of the data, with no consideration given to the variation of the dependent or independent variables. Hence, the design equations provided will not suffice, but will aid in determining the variation of t from the variations of X_i . In other words, if any of the basic (independent) variables is random, the dependent variable will likewise be random; its probability distribution, as well as its moments (mean and variance), will be functionally related and may be derived from those of the basic random variables using the best models (AASHTO equations).

In general, if t is a function of several variables,

$$t = g(X_1, X_2, \dots, X_n) \quad (10)$$

the exact moments of t may be obtained as the mathematical expectation of $g(X_1, X_2, \dots, X_n)$; hence, the mean or expected value of t , $E(t)$, and the variance, $\text{Var}(t)$, could be evaluated as follows:

$$\begin{aligned} E(t) &= \int_{-\infty}^{+\infty} \dots \int_{-\infty}^{+\infty} g(X_1, X_2, \dots, X_n) \\ &\quad \times f_{X_1, X_2, \dots, X_n}(X_1, X_2, \dots, X_n) \\ &\quad dX_1 dX_2, \dots, dX_n \end{aligned} \quad (11)$$

and

$$\begin{aligned} \text{Var}(t) &= \int_{-\infty}^{+\infty} \dots \int_{-\infty}^{+\infty} [g(X_1, X_2, \dots, X_n) - E(t)]^2 \\ &\quad \times f_{X_1, X_2, \dots, X_n}(X_1, X_2, \dots, X_n) dX_1 dX_2 \dots dX_n \end{aligned} \quad (12)$$

Here, $f_{X_1, X_2, \dots, X_n}(X_1, X_2, \dots, X_n)$ is the joint probability distribution function of X_1, X_2, \dots, X_n . Because the design inputs are independent

$$\begin{aligned} f_{X_1, X_2, \dots, X_n}(X_1, X_2, \dots, X_n) &= f_{X_1}(X_1) \\ &\quad \times f_{X_2}(X_2), \dots, f_{X_n}(X_n) \end{aligned} \quad (13)$$

where $f_{X_1}(X_1), f_{X_2}(X_2), \dots, f_{X_n}(X_n)$ are the probability distribution of X_1, X_2, \dots, X_n respectively.

Clearly, to evaluate the mean and variance of t with the above equations (Equations 11 through 13), information on $f_{X_1}(X_1), f_{X_2}(X_2), \dots, f_{X_n}(X_n)$ is needed. However, in many cases the density functions of X_1, X_2, \dots, X_n may not be known. Furthermore, even when such density functions are known, the integrations indicated above may be difficult to perform. For these reasons, approximate mean and variance of t would be practically useful and may be obtained as follows.

Expand the function $g(X_1, X_2, \dots, X_n)$ in a Taylor's series expansion about the mean. Assuming independence of X_1, X_2, \dots, X_n , the resulting expressions, if we truncate at the linear terms, are as follows:

$$\bar{t} = g(\bar{X}_1, \bar{X}_2, \dots, \bar{X}_n) \quad (14)$$

$$\sigma_t^2 = \sum_{i=1}^n \left(\frac{\partial t}{\partial X_i} \right)^2 \sigma_{X_i}^2 \quad (15)$$

where bars are used above the terms to indicate their means, and σ^2 represents the variance of the terms.

This method of approximating the mean and variance has been used extensively in design models (1,2,6) and was proven effective (within ± 10 percent of actual values) especially when the variance of X_i is small relative to $g(\bar{X}_1, \bar{X}_2, \dots, \bar{X}_n)$.

Evaluating the mean and variance of the design parameters is the key to applying Equations 14 and 15 to any pavement

design model that needs some consideration. The soil support value, for example, could vary greatly from point to point along the pavement. Estimating its mean and variance during the service life of the pavement is not an easy task. In addition, SSV will vary throughout the year depending on several factors, such as moisture. However, the variabilities in the design parameter could be assessed from past experience or from testing, if required. These variabilities are then combined, using Equations 14 and 15, to obtain \bar{t} and σ_t .

In the AASHTO model for flexible pavements, Equation 1, the pavement thickness is described by SN, which is a function of several variables; thus, by using Equation 14, the mean of SN can be written as follows:

$$\overline{\text{SN}} = f(\overline{W}, \overline{\text{SSV}}, \overline{R_f}, \overline{P_t}) \quad (16)$$

which is Equation 1 with bars over the terms to indicate the mean values.

Determination of the variance of SN is the next task. Using Equation 15, the variance of SN can be written as,

$$\begin{aligned} \sigma_{\text{SN}}^2 &= \left(\frac{\partial \text{SN}}{\partial W} \right)^2 \sigma_W^2 + \left(\frac{\partial \text{SN}}{\partial \text{SSV}} \right)^2 \sigma_{\text{SSV}}^2 \\ &\quad + \left(\frac{\partial \text{SN}}{\partial R_f} \right)^2 \sigma_{R_f}^2 + \left(\frac{\partial \text{SN}}{\partial P_t} \right)^2 \sigma_{P_t}^2 \end{aligned} \quad (17)$$

where the partial derivatives of SN (with respect to the design parameters) are evaluated at \overline{W} , $\overline{\text{SSV}}$, $\overline{R_f}$, and $\overline{P_t}$. If R_f and P_t are kept constant, that is, they do not vary their respective variance, $\sigma_{R_f}^2$ and $\sigma_{P_t}^2$ are zero, Equation 17 becomes

$$\sigma_{\text{SN}}^2 = \left(\frac{\partial \text{SN}}{\partial W} \right)^2 \sigma_W^2 + \left(\frac{\partial \text{SN}}{\partial \text{SSV}} \right)^2 \sigma_{\text{SSV}}^2 \quad (18)$$

Differentiating Equation 1 according to Equation 18 and substituting, the final result for the variance of SN is

$$\sigma_{\text{SN}}^2 = \left(\frac{1}{K_f} \right)^2 [(CV_W)^2 + 0.1452 \sigma_{\text{SSV}}^2] \quad (19)$$

where

$$\begin{aligned} K_f &= \frac{9.36}{\overline{\text{SN}} + 1} + \log 0.38(4.2 - P_t) \\ &\quad \times \left\{ \frac{5677.86(\text{SN} + 1)^{4.19}}{[0.4(\text{SN} + 1)^{5.19} + 1094]^2} \right\} \end{aligned}$$

and CV_W is the coefficient of variation of W .

With a similar approach, the mean and variance of the concrete slab thickness, D , for rigid pavements, are computed and the results are

$$\overline{D} = f(\overline{W}, \overline{S_c}, \overline{P_t}, \overline{k}) \quad (20)$$

and

$$\sigma_D^2 = \left(\frac{1}{K_r} \right)^2 \left[(CV_W)^2 + (CV_{S_c})^2 \left(\frac{4.22 - 0.32P_t}{2.3} \right)^2 \right] \quad (21)$$

where

$$K_r = \frac{7.35}{\bar{D} + 1} + \frac{13.74 \times 10^7 (\bar{D} + 1)^{7.46} \log 0.333(4.5 - P_i)}{[(\bar{D} + 1)^{8.46} + 1.624 \times 10^7]^2}$$

CV_W and CV_{S_c} are the coefficient of variation of W and S_c , respectively. Observe that Equation 20 is identical to Equation 2 with bars used over the expressions to represent the mean values.

Nomographic Solutions

The simplicity afforded by nomographic solutions in practical applications is appealing. For this reason, the expected variation in pavement thickness (flexible and rigid) is presented in a nomographic form. Figures 2 and 3 are solutions for Equations 16 and 19, respectively; whereas Figures 4 and 5 are solutions for Equations 20 and 21, respectively. Figure 6 compares values calculated by using the equations and values obtained from the nomographs. Clearly, the closeness of these points to the equality line easily verifies the accuracy of the nomographs.

Selecting Appropriate Reliability Based on Least-Cost Concept

Assuming that the pavement thickness is a normal variate with a known mean and variance (or standard deviation), the

reliability of a pavement thickness, t_a , can be evaluated by normal distribution tables. Conversely, given the reliability, the pavement thickness can be estimated. For example, consider that for certain conditions the mean slab thickness is 7 in. with a standard deviation of 0.4 in. In statistical terms, a 7-in. concrete slab will have a 50 percent reliability. In other words, if a 7-in. concrete slab is selected as a design thickness for the conditions given, 50 percent of such pavement will deteriorate early and reach its terminal serviceability before the intended design life. On the other hand, if the reliability is to be increased (say to 99 percent), the thickness should be $t = 7 + z_r(0.4)$. Using the normal distribution tables and for $R = 0.99$, $z_r = 2.33$, and thus $t = 8$ in.

The above example indicates that the design can represent an underdesign or overdesign, depending on the reliability value selected. Generally speaking, if the mean thickness is selected, about one-half the road will be underdesigned and one-half overdesigned. On the other hand, if a value corresponding to a high reliability is selected, most of the road will be overdesigned. Therefore, an optimal design will be of interest. Such a design will serve both statistical safety (reliability) and economy. For this purpose, the least-cost concept proposed by Yoder (13) is adopted.

The least-cost analysis just mentioned, suggests that the optimum design value (this term is adjusted to optimum pavement thickness in this paper) depends on the variability of the soil deposit and the traffic conditions in the site. Yoder presented several curves that relate percentile value (labeled as reliability here) for least cost as a function of soil variability (coefficient of variation), traffic, and unit cost of the pavement structure. These curves are modified and represented here in Figure 7. In this figure, cost includes both initial cost and

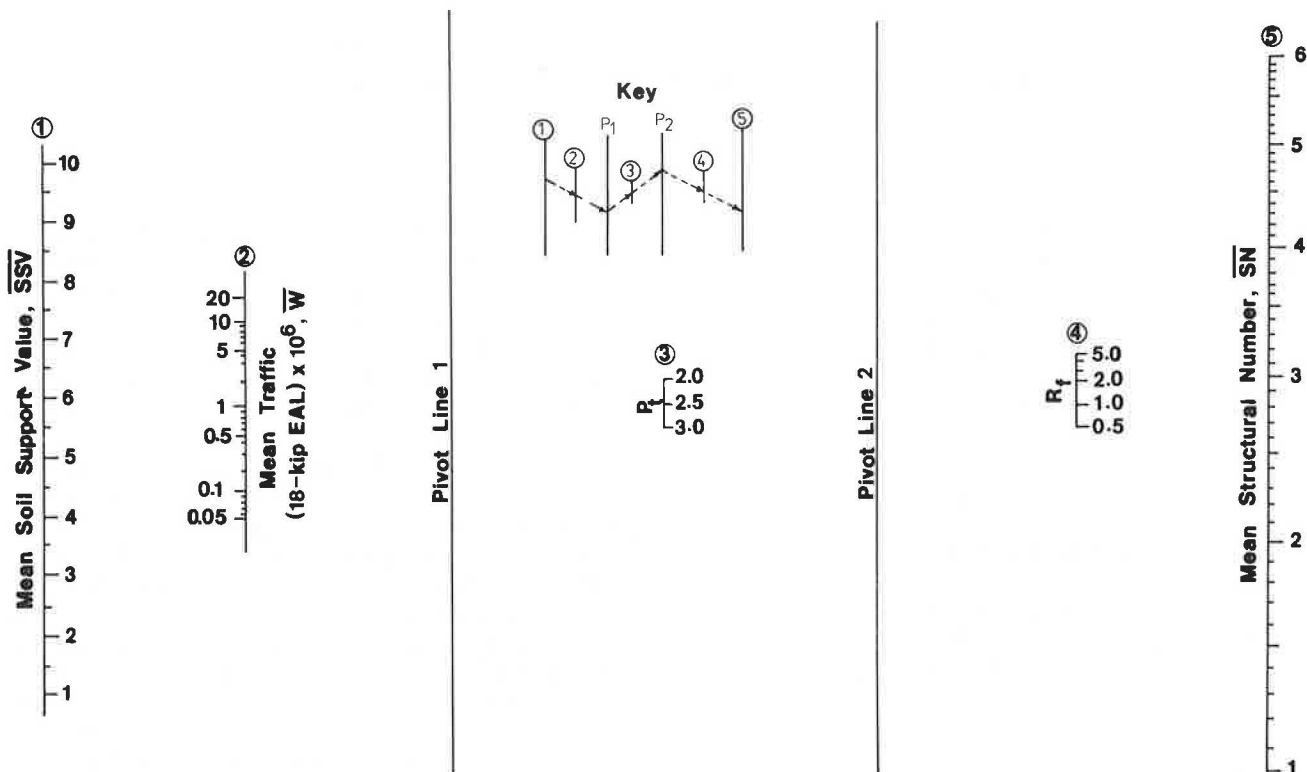


FIGURE 2 Nomograph for mean structural number, \overline{SN} .

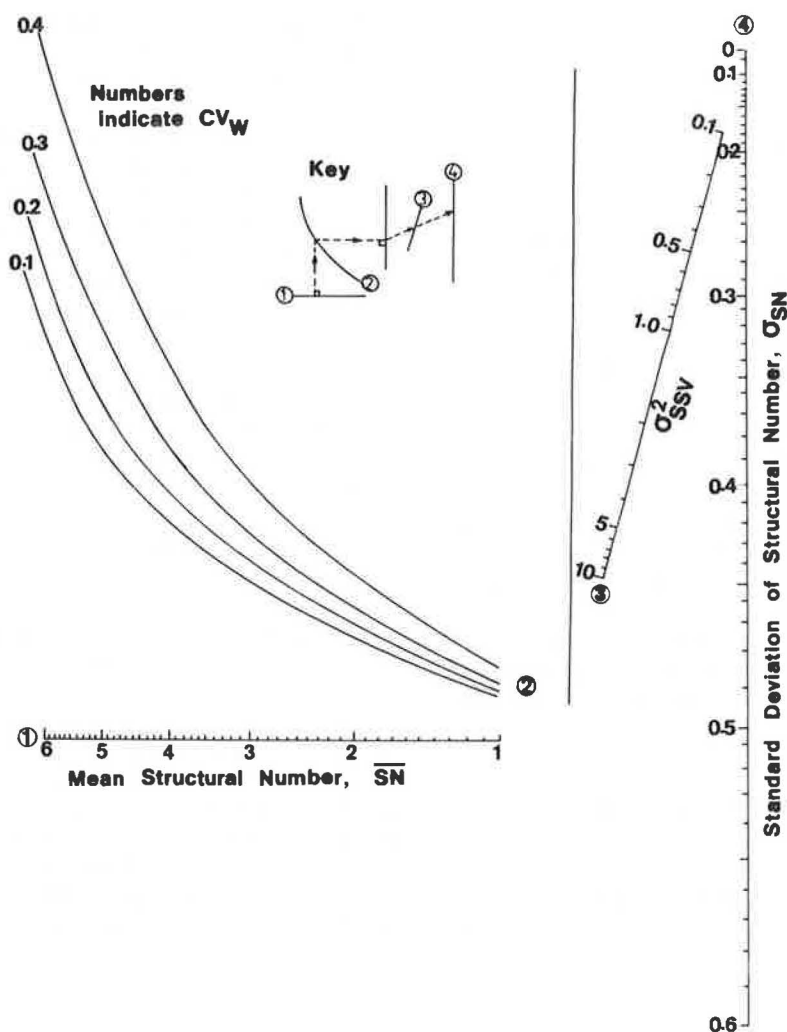


FIGURE 3 Nomograph for standard deviation of structural number, σ_{SN} .

maintenance cost needed to repair parts of the road that have been underdesigned. In addition, the cost ratio, CR, is defined as the ratio of the unit cost of maintenance to the unit initial cost. Estimates of this factor must be made to use this method. However, and based on the data presented, CR has a weighted average of 2, 3, and 4 respectively for roads with low ($< 10^5$ EAL), medium ($10^5 - 10^6$ EAL) and high ($> 10^6$ EAL) traffic. Therefore, by knowing traffic, CR could be approximated; and with the CV of soil deposit, the reliability of producing an optimum design can be evaluated.

Example of Statistical Pavement Design

This example is presented to illustrate the use of the statistical design charts. Consider a two-layered flexible pavement (asphalt concrete and gravel base) with the following input data:

Factor	Mean Value	CV	Variance
W, EAL	5×10^6	0.40	4.0×10^{12}
SSV	7	0.40	7.84
P_t	2.5	—	—
R_f	2.0	—	—

Enter Figure 2 with the mean value of the design factors to obtain the mean structural number, $\overline{SN} = 3.4$. With \overline{SN} and the other inputs, enter Figure 3 and obtain the standard deviation of structural number $\sigma_{SN} = 0.58$. With the traffic of 5×10^6 18-kip EAL, CR = 3, and $CV_{SSV} = 0.40$ (or 40 percent), obtain reliability for optimum design, $R = 84$ percent or the area bounded between $-\infty$ and z under standard normal curve is 0.84, thus, $z = 1.0$. Therefore, $SN = 3.4 + (1.0) \times (0.58) = 3.98$. This implies that, for the conditions given, a pavement with $SN = 3.4$ will have a reliability of 50 percent, whereas the optimum design of $SN = 3.98$ will have $R = 84$ percent. In other words, if we consider a 10-in. gravel base (layer coefficient $a_2 = 0.14$), this pavement will require a 6.1-in. asphalt concrete ($a_1 = 0.42$) for an optimum design with a reliability of 84 percent. If the AASHTO design were to be followed with no consideration given to the variations, a 4.8-in. asphalt concrete layer will suffice. However, this pavement will have only 50 percent reliability and, thus, will have a 50 percent chance of failure before its design life (20 years) ends, whereas the pavement with $R = 84$ percent will have only a 16 percent chance of failure.

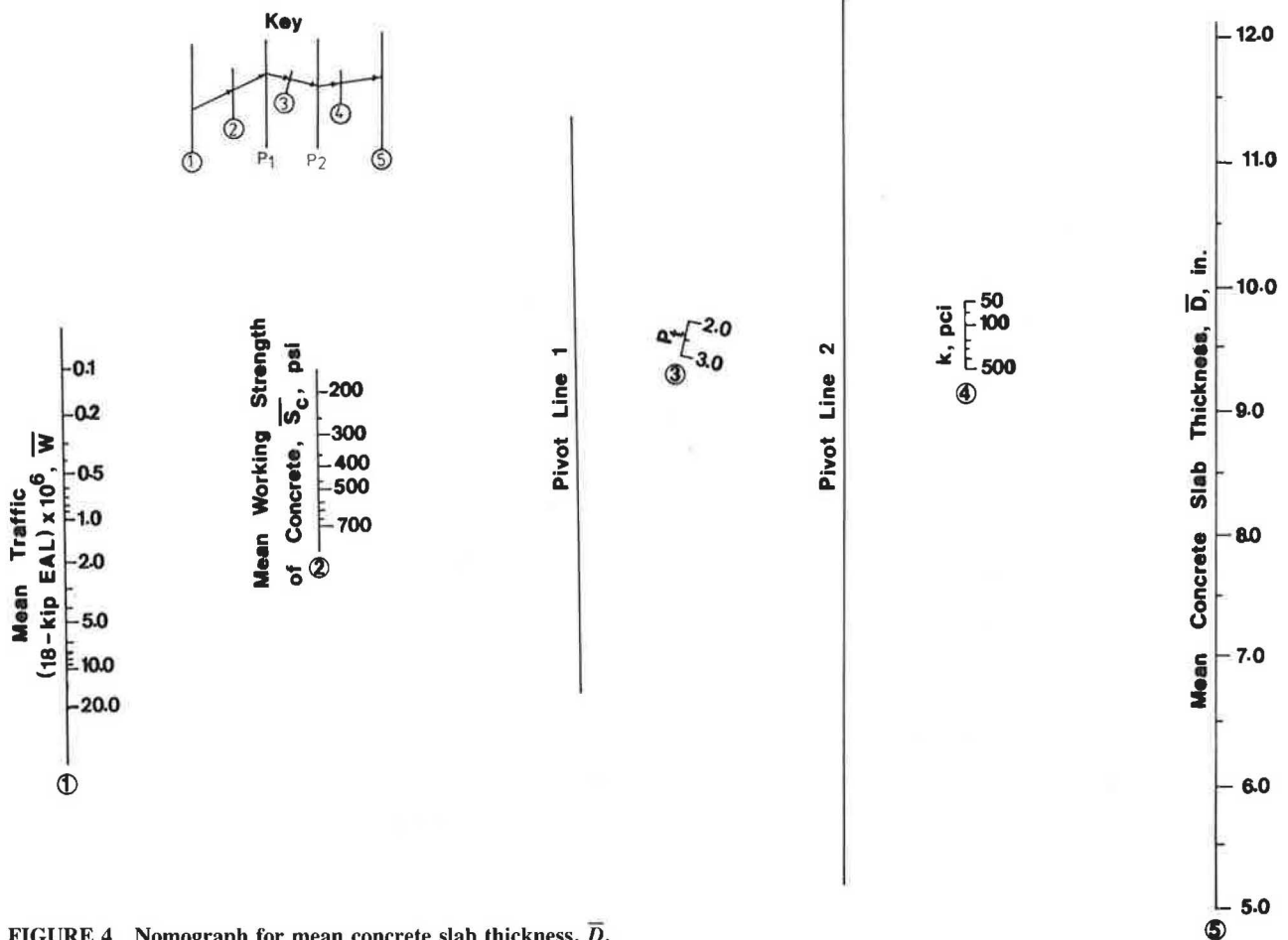


FIGURE 4 Nomograph for mean concrete slab thickness, \bar{D} .

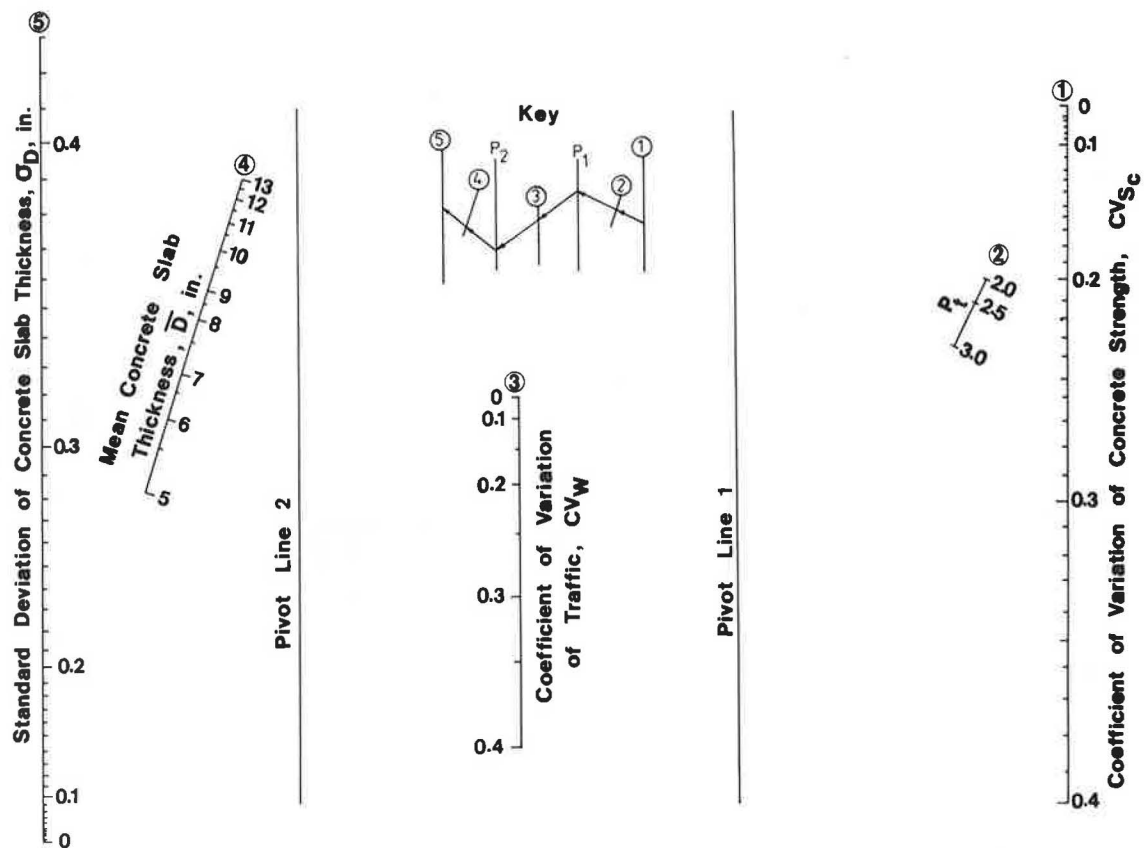


FIGURE 5 Nomograph for standard deviation of concrete slab thickness, σ_D .

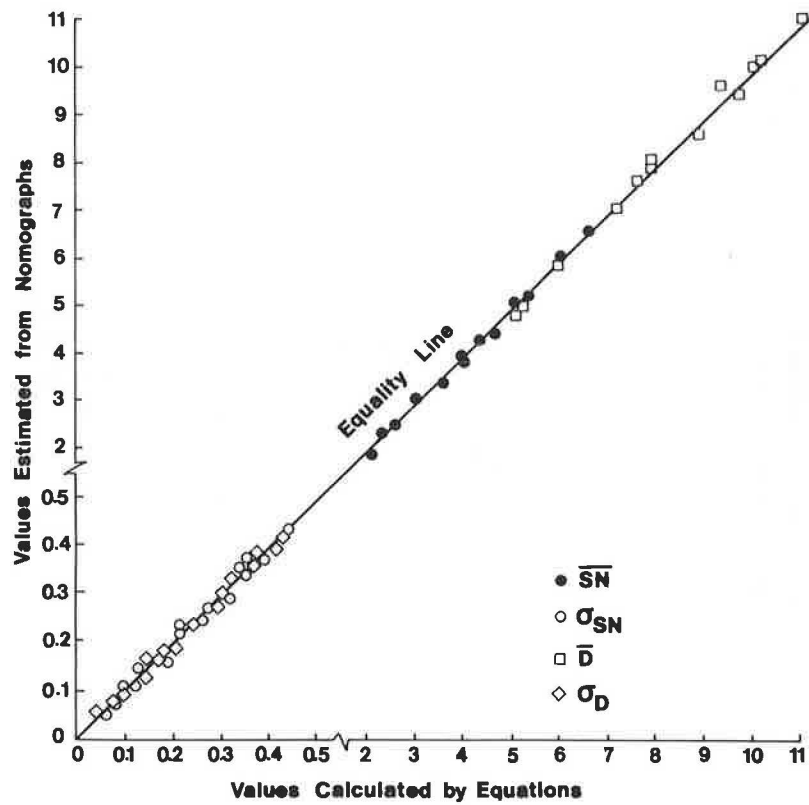


FIGURE 6 Comparisons among values, by equations and nomographs.

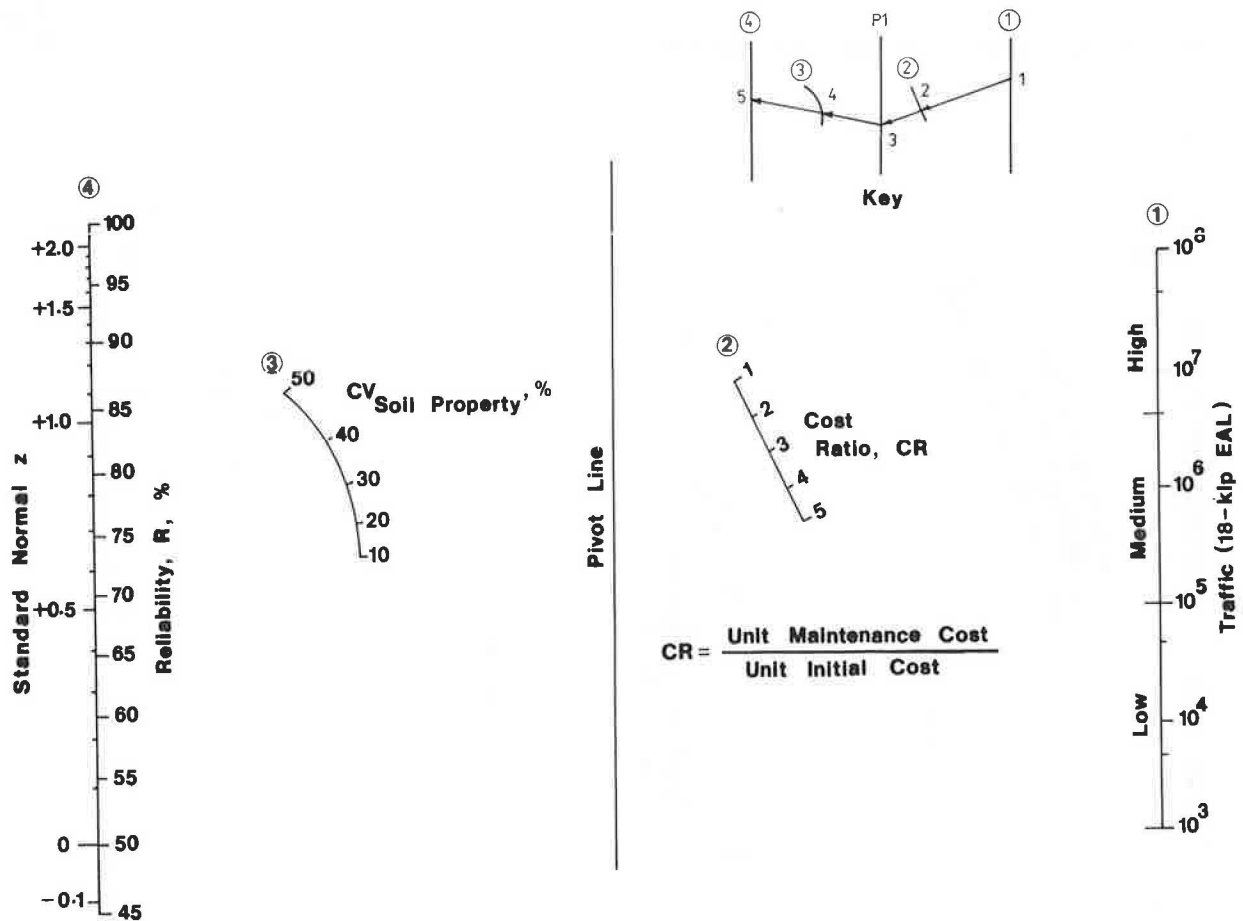


FIGURE 7 Reliability, R , for the least-cost design.

SUMMARY

Pavements, as do many other structures, possess random variabilities associated with almost all design parameters. Conventional pavement design models, in which the design factors are treated as deterministic quantities, would seem inadequate. In these cases, using a probabilistic approach accounts for the variabilities in design parameters, whereas statistical analysis quantifies their effects. However, strict reliance on results obtained by probabilistic and statistical methods should not be exercised but must rather be complemented by sound engineering judgment. Yet, applying probabilistic and statistical techniques to the analysis design of pavements, as has been done in this paper, provides the means for some important applications that were not previously possible. These include the following:

1. The ability to make the design process sensitive and adjustable for many variabilities and uncertainties in design parameters.
2. Providing standards for conducting design optimization; and
3. Affording the means for designing at different levels of reliability and, thus, design adequacy can be easily estimated.

REFERENCES

1. M. I. Darter, W. R. Hudson, and J. L. Brown. Statistical Variation of Flexible Pavement Properties and Their Consideration in Design. In *Proc., Association of Asphalt Paving Technologists*, Vol. 42, 1973, pp. 589-615.
2. M. I. Darter, B. F. McCullough, and J. L. Brown. Reliability Concepts Applied to the Texas Flexible Pavement System. In *Highway Research Record 407*, HRB, National Research Council, Washington, D.C., 1972, pp. 146-161.
3. M. I. Darter, W. R. Hudson, and R. C. G. Hass. Selection of Optimal Pavement Designs Considering Reliability, Performance, and Costs. In *Transportation Research Record 485*, TRB, National Research Council, Washington, D.C., 1974, pp. 67-79.
4. M. I. Darter and W. R. Hudson. *Probabilistic Design Concepts Applied to Flexible Pavement Design System*. Research Report 123-18. Texas Highway Department, Texas Transportation Institute, May 1973.
5. P. F. McManus and E. J. Barenberg. Impact of Subgrade Variability on Pavement Construction-Maintenance Cost Trade-offs. In *Special Report 148*, TRB, National Research Council, Washington, D.C., 1974, pp. 12-23.
6. P. K. Kher and M. I. Darter. Probabilistic Concepts and Their Applications to AASHO Interim Guide for Design of Rigid Pavements. In *Highway Research Record 466*, HRB, National Research Council, Washington, D.C., 1973.
7. M. I. Darter. Probabilistic Applications to Rigid Pavement Structure. Presented at the ASCE Specialty Conference on Probabilistic Methods in Engineering, Stanford University, Stanford, Calif., June 24-26, 1974.
8. J. L. Von Quintus, F. N. Finn, W. R. Hudson and F. L. Roberts. *Flexible and Composite Structures for Premium Pavements, Vol. 1, Development of Design Procedure*. Draft Report, FHWA-RD-80, Federal Highway Administration, December 1979.
9. *Interim Guide for Design of Pavement Structures*. AASHTO, Washington, D.C., 1981.
10. D. R. Benjamin and C. A. Cornell. *Probability, Statistics, and Decision for Civil Engineers*. McGraw-Hill, N.Y., 1970.
11. H. Findakly, F. Moavenzadeh, and J. Soussou. Stochastic Model of Pavement Systems. *Transportation Engineering Journal*, Vol. 100, No. TE 1, Feb. 1972, pp. 57-70.
12. E. J. Yoder and M. W. Witzczak. *Principles of Pavement Design*, 2nd ed. Wiley, N.Y., 1975.
13. E. J. Yoder. Selection of Soil Strength Values for the Design of Flexible Pavements. In *Highway Research Record 276*, HRB, National Research Council, Washington, D.C., 1969.
14. J. A. Deacon and R. L. Lynch. *Determination of Traffic Parameters for the Prediction, Projection, and Computation of EWLS*. Final Report KYHPR-64-21, Kentucky Highway Department, 1968.
15. *Manual of Instructions, Part 8: Materials*. Utah State Department of Highways, n.d.
16. *The Repeatability of Test Results Using Various California Bearing Ratio Procedures and Resistance R-Value*. Utah Research Report 500-908, Utah State Department of Highways, Materials and Testing Division, Utah Project HPR-1 (5), August 1967.
17. C. J. Van Til, B. F. McCullough, B. A. Vallerger, and R. G. Hicks. *Evaluation of AASHO Interim Guides for Design of Pavement Structures*. NCHRP Report 128, HRB, National Research Council, Washington, D.C., 1972.
18. G. B. Sherman. In Situ Materials Variability. In *Special Report 126*, HRB, National Research Council, Washington, D.C., 1971, pp. 180-188.

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