Reliability of Pavement Structures using Empirical-Mechanistic Models

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ABSTRACT

The National Cooperative Highway Research Program has undertaken a sizeable research effort to develop a mechanistic-empirical design procedure for pavement structures: NCHRP 1-37A. This procedure will likely replace the current AASHTO empirical design method, which has been in use since the early 1970s. However, this change may take many years to materialize and become widely accepted. One of the main improvements is the use of axle load spectra instead of an aggregate measure of mixed traffic, such as the number of equivalent single axle loads (ESALs). This change will considerably reduce the uncertainty in the pavement performance estimations. In their present format, both the current AASHTO design method and the forthcoming mechanistic procedure incorporate ad hoc procedures to assess design reliability. The objective of this study is to develop and evaluate an alternative approach to assess this reliability, and to highlight some important aspects that are often ignored.

Using Monte Carlo simulation techniques, the performance of a pavement structure designed with the 1993 Guide with various levels of reliability under different traffic volumes is evaluated. Effects of the environment, structural strength, and traffic volume on pavement reliability and performance are discussed. The load models employed include consideration of the distribution of loads by the actual load spectra measured in field. The reliability analysis results permit discussion related to the implied reliability of the current AASHTO design method. An approach to make the AASHTO design more robust is suggested. Another useful outcome of the present study is the quantification of the relative influence on reliability of variables defining the loading as well as the capacity. These findings might help identify critical areas to which resources might be allocated to improve pavement reliability.

INTRODUCTION

The current AASHTO design equation for the design of flexible pavement structures *(1)* is primarily based on the results of the AASHO Road Test, which took place in Ottawa, Illinois in the late 1950s to early 1960s *(2)*. The AASHO Committee on Design first published an interim design guide in 1961. It was revised in 1972 *(3)* and 1981. In 1984-85, the Subcommittee on Pavement Design and a team of consultants revised and expanded the guide under NCHRP project 20-7/24 and issued the current guide in 1986 *(1)*. The results of the AASHO Road test were used to develop a deterioration model that has provided the basis for flexible and rigid pavement design.

The original mathematical model chosen for both the flexible and rigid pavement analysis is given by Equation 1.

$$p_t = c_0 - (c_0 - c_1)\left(\frac{W_t}{\rho}\right)^{\beta} \tag{1}$$

p_t : serviceability value at time t ($c_1 \le p_t \le c_o$)

c_0 : initial serviceability value

c_1 : serviceability level at which a test section was considered to have failed

W_t : accumulated axle load applications at the time t

β, ρ : regression parameters or functions.

Serviceability is defined as the ability of a specific section of pavement to serve traffic in its existing conditions *(4)*. Equation 1 estimates p_t in terms of Present Serviceability Index (PSI), defined as a mathematical combination of values obtained from certain distress measurements so formulated as to predict the Present Serviceability Rating (PSR) for those pavements within prescribed limits *(4)*. PSR is the mean of the individual ratings (by individuals of a specific panel) of the present serviceability of a specific section of roadway. The individual ratings varied between 5 (Excellent) to 0 (Very Poor). Equation 2 was used by AASHO to determine PSI *(2)*.

$$p_t = 5.03 - 1.91\log\left(1 + \overline{SV}\right) - 0.01\sqrt{C + P} - 1.38\overline{RD}^2 \tag{2}$$

$\overline{SV}$: mean of the slope variance

C : linear cracking

P : patching area

$\overline{RD}$: average rut depth

β and ρ are given by Equations 3 and 4.

$$\beta = 0.4 + \frac{B_o (L_1 + L_2)^{B_2}}{(D+1)^{B_1} L_2^{\ B_3}} \tag{3}$$

$$\rho = \frac{A_0 (D+1)^{A_1} L_2^{\ A_3}}{(L_1 + L_2)^{A_2}} \tag{4}$$

$$D = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{5}$$

D_1 : surface thickness (in)

D_2 : base thickness (in)

D_3 : subbase thickness (in)

L_1 : nominal axle load in kips

L_2 : axle type (1 for single axles and 2 for tandem axles)

a_i : layer coefficients.

It was found that designs failing early tended to have an increasing rate of serviceability loss, while more adequate designs as a rule had a decreasing loss rate. The function ρ is equal to the number of load applications at which $p_t = 1.5$ (failure condition). The regression equations for ρ, β and p_t were obtained using a stepwise regression approach, which did not account for unobserved events. In addition, there are serious inconsistencies in the specification of the regression equations for β and ρ. These two aspects have led to the estimation of biased regression parameters. Finally, all the models are intrinsically linear leading to unnecessary large regression errors. The estimated standard error of Equation 1 was approximately 0.707 PSI *(5)*.

Current AASHTO design equations

After estimating the regression parameters, Equations 6 and 7 were developed for flexible pavements *(2)*.

$$\beta = 0.4 + \frac{0.081(L_1 + L_2)^{3.23}}{(D+1)^{5.19} L_2^{\ 3.23}} \quad (6)$$

$$\log \rho = 5.93 + 9.36 \log(SN+1) - 4.79 \log(L_1 + L_2) + 4.33 \log(L_2) \quad (7)$$

SN : structural number of the pavement given by Equation 8

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \quad (8)$$

All other variables are previously defined.

The procedure is simplified if an equivalent 18 kip (80 kN) single axle load is used. By combining Equations 1, 6 and 7 and setting L_1 = 18 and L_2 = 1, Equation 9 is obtained.

$$\log W_{t18} = 9.36 \log(SN+1) - 0.2 + \frac{\log\left(\frac{4.2 - p_f}{4.2 - 1.5}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} \quad (9)$$

W_{t18} : number of 18 kip (80 kN) single axle load applications to time t

p_f : the terminal serviceability index.

Equation 9 is applicable only to flexible pavements in the AASHO Road Test with an effective subgrade resilient modulus of 3,000 psi (20.7 MPa).

The original equations were developed under a given climatic setting with a specific set of pavement materials and subgrade soils. The climate at the test section was temperate with an average annual precipitation of about 34 in (864 mm). The average depth of frost penetration was about 28 in. (711 mm). The subgrade soils consisted of A-6 and A-7-6 that were poorly drained with CBR values ranging from 2 to 4.

For other subgrade and environmental conditions, Equation 9 was later modified to Equation 10

$$\log W_{t18} = 9.36 \log(SN+1) - 0.2 + \frac{\log\left(\frac{4.2 - p_f}{4.2 - 1.5}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log(M_R) - 8.07 \quad (10)$$

M_R = the effective roadbed soil resilient modulus. To take local precipitation and drainage conditions into account, Equation 8 was modified to Equation 11.

$$SN = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 \tag{11}$$

m_2 : drainage coefficient of base course.

m_3 : drainage coefficient of subbase course.

Equation 10 is the performance equation which gives the allowable number of 18 kip single axle load applications W_{t18} to cause the reduction of PSI to p_f. If the predicted number of applications W_{t18} is equal to W_{18} (expected traffic in ESALs), the reliability of the design is only 50% because all variables in Equation 10 predicts pavement performance conditional on mean values of the design variables. To achieve a higher level of confidence in the designs, W_{18} must be smaller than W_{t18} by a normal deviate Z_R given by Equation 12.

$$Z_R = \frac{\log W_{18} - \log W_{t18}}{S_0} \tag{12}$$

Where Z_R is the normal deviate for a given reliability R and S_0 is overall standard deviation, which accounts for the variability of all variables. Combining Equations 10 and 12 and replacing $(4.2 - p_f)$ by ΔPSI yields Equation 13, which is the current final design equation for flexible pavements.

$$\log W_{t18} = Z_R S_0 + 9.36 \log(SN + 1) - 0.2 + \frac{\log\left(\frac{4.2 - p_f}{4.2 - 1.5}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log(M_R) - 8.07 \tag{13}$$

The DNPS86 computer program issued by AASHTO can be used to solve Equation 13 along with the nomograph provided by AASHTO 1993. Table 1 shows the percentage distribution of overall variance when the AASHTO equations are used for pavement design as provided by AASHTO 1986. Applications of the reliability concept require the selection of a standard deviation that is representative of local conditions. It is suggested that standard deviations of 0.45 be used for flexible pavements and 0.35 for rigid pavements *(4)*.

Non-linear Model

In order to address some of the shortcomings identified with the original AASHTO deterioration model (Equation 1), on which the current AASHTO design equation is based (Equation 13), a more accurate and statistically sound nonlinear deterioration model has been recently developed *(5)*. This model was developed using the same data set and the same number explanatory variables, while relaxing the linear specification restriction and accounting for unobserved events. The model also employed a simultaneous estimation approach rather than a stepwise approach. A random effects approach was used for the estimation of the regression parameters resulting in an estimated combined standard error of 0.377 PSI, which is approximately half the standard error of the original model. By addressing these problems, the parameter estimates are unbiased.

The improved incremental nonlinear model describing serviceability, p_t in terms of PSI (present serviceability index) at time t is given as follows:

$$p_t = \beta_1 + \beta_2 e^{\beta_3 H_1} + (1 + \beta_4 H_1 + \beta_5 H_2 + \beta_6 H_3)^{\beta_7} \sum_{l=1}^{t} e^{\beta_8 G_l} N_{l-1}^{\beta_9} \Delta N_l + \varepsilon \qquad (14)$$

Where H_1, H_2, and H_3 have the same meanings as D_i in Equation 11; G_l is the frost gradient which represents the change in frost depth with time over a two-week interval, l; ε represents the model error term; ΔN_l is the incremental "equivalent" traffic for the two-week interval, l; and N_{l-1} is the cumulative equivalent traffic up until the interval, $l-1$. β_i represent the regression parameters.

Further details relating to the performance model of Equation 14 are included in the following discussion of the reliability analysis of a pavement structure designed per AASHTO specifications. Equation 14 is not intended to replace the original AASHTO design equation but rather to demonstrate the impact on design and reliability of using models with a reduced standard error.

Finally it should be emphasized that the nonlinear equation was developed based on the same data set and used the same number of variables, yet the standard error was half of the original one. Halving the prediction error could be translated into significantly more accurate prediction, resulting in improved resource allocation and cost reduction.

RELIABILITY ANALYSIS OF A PAVEMENT STRUCTURE DESIGNED PER AASHTO SPECIFICATIONS

The reliability of a specific pavement structure was investigated using the performance model given by Equation 14. The objective was to assess the actual reliability of the pavement structure which is designed with a specific reliability according to AASHTO 1993 method.

The reliability of this AASHTO-designed pavement is assessed for a location where a deterministic description of the frost gradient, G_l (in/day) in Equation 14 is assumed and a simplified variation over each year (26 two-week intervals) is employed as follows:

$$G_l = \begin{cases} 0, & 0 \le l \le 4 \\ 4, & 4 < l \le 8 \quad \text{(frost depth increasing)} \\ -8, & 8 < l \le 13 \quad \text{(frost depth decreasing)} \\ 0, & 13 < l \le 26 \end{cases} \tag{15}$$

These mean values of G_l were estimated from the data of the AASHO Road Test. The designed pavement structure is required to have thicknesses of the surface, base, and subbase (i.e., H_1, H_2, and H_3) equal to 4 in., 8 in., and 10 in., respectively. A normal distribution is employed for each thickness with an assumed coefficient of variation (COV) of 10% for the AC and base layer and 12.6% for the subbase layer based on research by Darter et al. *(6)*. The thicknesses selected result in a structural number (SN) approximately equal to four.

For simplicity the design traffic was considered as composed of trucks corresponding to Class 9 ("*18 wheelers*") in FHWA scheme as this trucks class causes the maximum damage to the pavement. The trucks were assumed to have 1 steering axle and 2 tandem axles. The traffic load spectra obtained from Site D502 located in the south west of Seguin on IH 10 was used as shown in Figure 1. Since the analysis is actually based on axle load spectra, it can be readily extended to all traffic classes. The incremental equivalent traffic, ΔN_l, for interval l is estimated as follows:

$$\Delta N_l = \left\{ \left[\sum_j m_{0j} \left(\frac{FA_j}{\beta_{10} \cdot 18} \right)^{\beta_{12}} \right] + \left[\sum_j m_{2j} \left(\frac{TA_j}{\beta_{11} \cdot 18} \right)^{\beta_{12}} \right] \right\} \Delta N \tag{16}$$

Where FA_j is the front (steering) axle load for load group j while m_{0j} is the number of axles in that front axle load group and TA_j is the tandem axle load for load group j while m_{2j} is the

number of axles in that load group. The number of axles for each load group is for a two-week interval. ΔN is the truck traffic for every two weeks. The cumulative equivalent traffic in Equation 14 is given as follows:

$$N_{l-1} = \sum_{q=1}^{l-1} \Delta N_q \tag{17}$$

The regression parameters, $\beta_1 - \beta_{12}$, in Equations 14 and 16 that were estimated using a random effects approach are listed in Table 2.

Information on the various random variables required for the simulation-based reliability analysis is summarized in Table 3. Note that the model error term, ε, in the performance deterioration model of Equation 14 is modeled as normal random variable with a zero mean and a standard deviation of 0.377. This is consistent with the assumptions of the classical regression model used for the development of the model.

EXAMPLE OF SUMULATED LOADING AND PERFORMANCE

A terminal value of 2.5 is considered with the serviceability model of Equation 14. Thus, when p_t falls below 2.5, the pavement is assumed to have failed. Monte Carlo simulations for a duration of ten years for the different levels of design reliability are conducted using the random variable information in Table 3 in combination with the performance model of Equation 14. The analysis focuses primarily on determining how different the predicted reliability of an AASHTO-designed pavement structure is from the reliability based on the more accurate nonlinear model. The influence of various parameters related to loading, climate, and structure on the performance and reliability of the selected pavement structure are also assessed. Three different cases of design reliability corresponding to 50%, 80% and 90% were considered for the analysis.

Loading

Table 4 shows the different cumulative traffic volumes applied on the pavement over the duration of design life corresponding to different levels of reliability and different values of S_0.

Performance

As an example, Figure 2 shows 100 realizations of the serviceability, p_t, as a function of time for the AASHTO design reliability of 50% and $S_0 = 0.5$. For a terminal serviceability, p_f, of 2.5, it is seen that a fraction of the pavements fail within ten years. An expanded discussion about pavement performance based on a larger number of simulations is presented in the following section. It is worthy noting the steps observed in the performance predictions, which correspond to the thawing periods.

RESULTS

The performance of the selected pavement structure was evaluated using Monte Carlo simulations. Table 5 depicts the number of simulations carried out and the corresponding reliabilities of the pavements for the various cases considered. The number of simulations required is stated with a confidence level of 95% and an error range of ±1%.

AASHTO recommends values of S_0 from 0.4 to 0.5 *(1)*. As is evident from Table 5, the actual reliabilities obtained for this range are very high as compared to the design reliabilities. This large discrepancy between AASHTO's implied reliability for the chosen design and the simulation-based reliability estimate is attributed to the fact that a more accurate performance model is used here in conjunction with a more accurate traffic characterization. Recall that the estimated standard error of the original AASHTO design equation was 0.707 PSI, while the standard error for the nonlinear model in Equation 14 is 0.377 PSI. From the above, it would appear that the approach incorporated in the AASHTO design procedure is conservative or, equivalently, that AASHTO designs may be more reliable than the procedure suggests.

Figure 3 demonstrate that the simulations with the nonlinear model capture the pavement behavior efficiently. As the design reliability is increased for the pavement with the same S_0, the number of pavement failures in the early years should reduce and those in the later years should increase, which is consistent with the results obtained as depicted in Figure 3. It is also evident from the simulation results that a reduction in the value of S_0 causes the actual reliability to be closer to the design reliability. It should be noted that changing the value of S_0 also changes the

design traffic for the pavement. It was found that a value of 0.2 for S_0 (much lower than the range suggested by AASHTO) would be better as it reduces the gap between actual and design reliabilities as determined by the simulation and depicted in Table 5. Thus changing the value of S_0 can be a way by which the AASHTO equation can be made robust and accurate.

EFFECT OF DESIGN VARIABLE ON PERFORMANCE

Aggregated Effect on Time to Failure based on Regression Analysis

In order to establish which design variables exert the largest influence on pavement performance, a regression analysis was carried out to estimate time to failure (T_f) of the pavement structures (that actually failed) as a function of the design variables. The following second-order surface was used to fit the data:

$$T_f = \alpha_0 + \alpha_1 Z_{H_1} + \alpha_2 Z_{H_2} + \alpha_3 Z_{H_3} + \alpha_4 Z_\varepsilon + \alpha_5 Z_{H_1}^2 + \alpha_6 Z_{H_2}^2 + \alpha_7 Z_{H_3}^2 + \alpha_8 Z_\varepsilon^2 + \alpha_9 Z_{H_1} Z_{H_2} + \alpha_{10} Z_{H_2} Z_{H_3} + \alpha_{11} Z_{H_3} Z_\varepsilon + \alpha_{12} Z_{H_1} Z_\varepsilon + \alpha_{13} Z_{H_2} Z_\varepsilon + \alpha_{14} Z_{H_1} Z_{H3} \quad (19)$$

Where the variables, $Z_{(.)}$, represent the number of standard deviations from their respective means of the variables, H_1, H_2, H_3 and ε (all normal), as described before. The parameter estimates ($\alpha_0 - \alpha_{14}$) are given in Table 6 along with their t-statistics for the design reliability of 50% and $S_0 = 0.5$. By expressing the values of the random variables in terms of standardized deviations from their means, the parameter estimates in Table 6 make comparison of the influence of each variable on pavement performance easier.

Based on the parameter estimates given in Table 6, the first-order effects of the variables can be summarized by three broad statements:

1. The variables, H_1 (surface thickness) and ε (model error), have the largest influence on pavement performance. If the surface thickness is sampled one standard deviation below its mean it will, on the average, cause a reduction in pavement life of approximately three years. On the other hand, if the model error is sampled one standard deviation below its mean it will, on the average, cause a reduction in pavement life of approximately five years

2. The variables H_2 and H_3 (representing base and subbase thicknesses, respectively) are clearly seen to have the smallest effect of performance. When they take on values that are one standard deviation below their means they reduce the pavement life by approximately one and a half years, on average.

Of the ten second-order terms in Equation 19, the largest second-order influence results from the product terms involving surface thickness, H_1, and model error, ε. Similar results were obtained for all the cases considered as depicted in Table 4.

The findings from these simulation studies suggest that, to increase pavement life, placing stricter quality control on the surface thickness might pay off twice as much, on average, than controlling the base and subbase thicknesses. However, the model error is seen to be almost as important. This suggests that efforts directed towards establishing accurate performance models might translate directly into better predictions of time to failure and reliability.

Analysis of Early and Late Failures

Once the relative effects on performance of the pavement structure were assessed for the various design variables, a more in-depth analysis was carried out to determine which variables were responsible for early failures and which for later ones. Early failures are usually attributed to poor construction practices without any further quantification or understanding of what causes these failures.

The 27,336 failed pavements were grouped into ten categories according to the year in which they failed for the case of design reliability of 50% and S_0 value of 0.5. Within each group, the average of each of the design variables was determined. The averages of these various design variables (expressed as standardized deviations from their mean values) are represented in Figure 4 for years 1 to 10.

Figure 4 reveals that for the failed pavements, two random variables show relatively uniform variation with time in their averages. For example, the failed pavements had base and subbase thicknesses (H_2 and H_3) on the order of half a standard deviation below their respective mean

values for all failed pavements uniformly over the ten years. Two variables, surface thickness (H_1) and model error (ε), however, displayed systematic variations with time. Considering the model error first, it can be seen that the early failures seem to result when the model error is very far below its mean (as many as 2 to 3 standard deviations on average in the first three years). Also, in the early years, surface thickness (H_1) among failed pavements was generally between 1 and 1.5 standard deviations below its mean value. Note however that among these early failures, it was generally the case that only one of these two random variables needed to be sampled significantly below its mean value for a failure to result. To explain this further, for the 1,106 early failures (or approximately 4.1% of all failures) where the time to failure, T_f, was less than 3 years, the sample correlation coefficient between ε and H_1 was large but negative. This is show graphically in Figure 5.

Although a very small number of pavements failed in the early years, these failures may be attributed to limited understanding of the inherent variability in pavement performance as reflected by performance model error. Otherwise, early failures may be generally attributed to lower surface thickness values than recommended.

As can be seen in Figure 4, in later years, the importance of the model error (ε) is diminished greatly. It is seen to have a noticeable trend with time. Late failures are generally less sensitive to modeling uncertainty. The surface thickness, H_1, is also seen to be of less importance among the later failures than it was for early failures. To contrast early failures with later ones in the light of the most important source of variability (model uncertainty) as well as to point out the importance of the frost gradient term, G_l, one might examine Figure 6. It may be seen as expected that there are systematic seasonal clusters of failures that occur over the five years shown in the figure. This characteristic results from the effect of the deterministic frost gradient modeled annually according to Equation 15, and reflects what was observed at the AASHO Road Test.

Finally, it may be noted that the rate of occurrence of failures increases with time as the traffic load cycles compromise the pavement's performance. Similar results were obtained for the rest of the cases considered.

CONCLUSIONS

Using simulation techniques, the reliability of a selected pavement structure has been studied. The design for this structure was based on the current AASHTO design approach and was designed with 50%, 80% and 90% reliability with overall standard deviations varying from 0.1 to 0.5. Results from the simulation studies that employed a nonlinear performance model with reduced standard error suggest that the reliabilities were higher for the values of standard deviation recommended by AASHTO, implying that the AASHTO design approach might be overly conservative when axle load spectra are used. An overall standard deviation of around 0.2 could be used to reduce the gap between the design and actual reliabilities, especially in those cases where accurate expected traffic information is available. It was also demonstrated that the simulations capture the pavement behavior satisfactorily.

A detailed examination of the failed pavements showed that the parameters that influence pavement performance to the greatest extent are the surface asphalt thickness and the model error. This supports the idea that most simulation approaches that do not account for model error are ignoring an important component of the overall performance variability.

When time to failure is studied, it is found that early failures may be attributed to either model error (ε) or to surface thickness (H_1) – sampling of these random variables two or more standard deviations below their means often caused early failures. Since model error is inherent to the modeling process and can not be avoided, during the construction process all possible resources should be utilized to control the variability of the thickness of the surface layer.

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Table 1 Percentage Distribution of overall variance

Type of prediction	Source of Variance	Value	Flexible pavement	Value	Rigid pavement
Traffic prediction	Traffic factor		14%		22%
	Unexplained		3%		4%
	Lack of fit		1%		1%
	Total Variance	0.0429	18%	0.0429	27%
Performance Prediction	Design Factor		45%		42%
	Unexplained		5%		8%
	Lack of fit		32%		23%
	Total Variance	0.1938	82%	0.1128	73%
Overall Variance		0.2367	100%	0.1557	100%

Table 2 Estimates of Parameters in Nonlinear Model based on a Random Effects Approach.

Parameter	Random Effects Estimate
β_1	4.24
β_2	-1.43
β_3	-0.856
β_4	1.39
β_5	0.329
β_6	0.271
β_7	-3.03
β_8	-0.173
β_9	-0.512
β_{10}	0.552
β_{11}	1.85
β_{12}	4.15

Table 3 Random Variables included in the Simulation Studies.

Random Variable	Distribution	Parameters
H_1	Normal	Mean = 4 in. (100 mm), CoV = 10%
H_2	Normal	Mean = 8 in. (200 mm), CoV = 10%
H_3	Normal	Mean = 10 in. (250 mm), CoV = 12.6%
ε	Normal	Mean = 0, std. dev. = 0.377
ΔN	Normal	CoV = 15%

Table 4 Traffic volume applied on the Pavement (design life of 10 years)

AASHTO Reliability (%)	So	ESALs applied over design life	Total No. of Trucks
50	0.10	1,047,300	965,198
80	0.10	862,798	795,160
90	0.10	779,679	718,557
50	0.15	1,047,300	965,198
80	0.15	783,119	721,727
90	0.15	672,726	619,988
50	0.20	1,047,300	965,198
80	0.20	710,799	655,077
90	0.20	580,445	534,942
50	0.25	1,047,300	965,198
80	0.25	645,158	594,582
90	0.25	500,822	461,561
50	0.40	1,047,300	965,198
80	0.40	482,417	444,599
90	0.40	321,700	296,481
50	0.50	1,047,300	965,198
80	0.50	397,430	366,274
90	0.50	239,494	220,719

Table 5 Monte Carlo Simulations for the Pavement

AASHTO Reliability (%)	So	Number of simulations	Number of failures	Actual Reliability (%)	Actual Simulations required
50	0.1	93,000	27,336	70.6	92,279
80	0.1	160,000	32,140	79.9	152,827
90	0.1	300,000	49,414	83.5	194,813
50	0.15	93,000	27,336	70.6	92,279
80	0.15	250,000	41,230	83.5	194,521
90	0.15	300,000	35,319	88.2	287,889
50	0.2	93,000	27,336	70.6	92,279
80	0.2	300,000	39,737	86.7	251,610
90	0.2	475,000	39,471	91.7	423,887
50	0.25	93,000	27,336	70.6	92,279
80	0.25	350,000	37,398	89.3	320,947
90	0.25	638,500	36,353	94.3	636,318
50	0.4	93,000	27,336	70.6	92,279
80	0.4	790,000	40,735	94.8	706,610
90	0.4	2,250,000	39,936	98.2	2,125,946
50	0.5	93,000	27,336	70.6	92,279
80	0.5	1,250,000	38,704	96.9	1,202,282
90	0.5	5,000,000	39,462	99.2	4,829,051

Table 6 Parameter estimates and corresponding t statistics for a second-order response surface for time to failure (n= 26,866; R^2 = 0.988; standard error = 5.77).

	Non-standardized Coefficients		Standardized Coefficients	t
	B	Std. Error	Beta	
(Constant)	348.2	0.196		1774.1
H_1	83.5	0.155	1.468	540.1
H_2	36.5	0.104	0.678	352.9
H_3	47.8	0.114	0.884	420.0
E	132.3	0.225	1.902	588.7
H_1^2	4.5	0.039	0.140	116.2
H_2^2	0.9	0.028	0.025	32.9
H_3^2	1.6	0.030	0.045	52.6
e^2	9.7	0.064	0.303	151.4
H_1 H_2	4.4	0.048	0.092	93.0
H_2 H_3	2.4	0.041	0.047	58.0
H_3 e	13.7	0.067	0.307	203.0
e H_1	23.1	0.084	0.511	275.6
e H_2	10.4	0.063	0.235	164.7
H_1 H_3	5.9	0.050	0.122	117.6

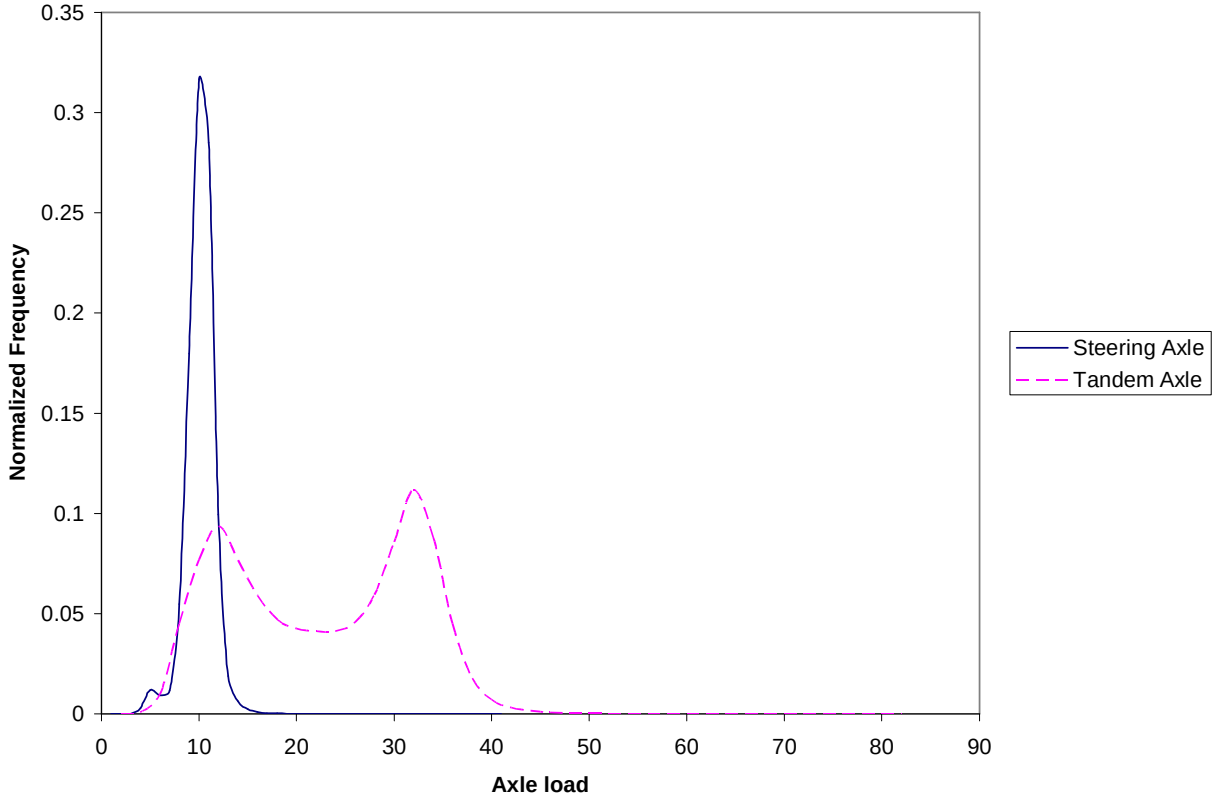

Figure 1 Load Spectra (in kips) as obtained from the field

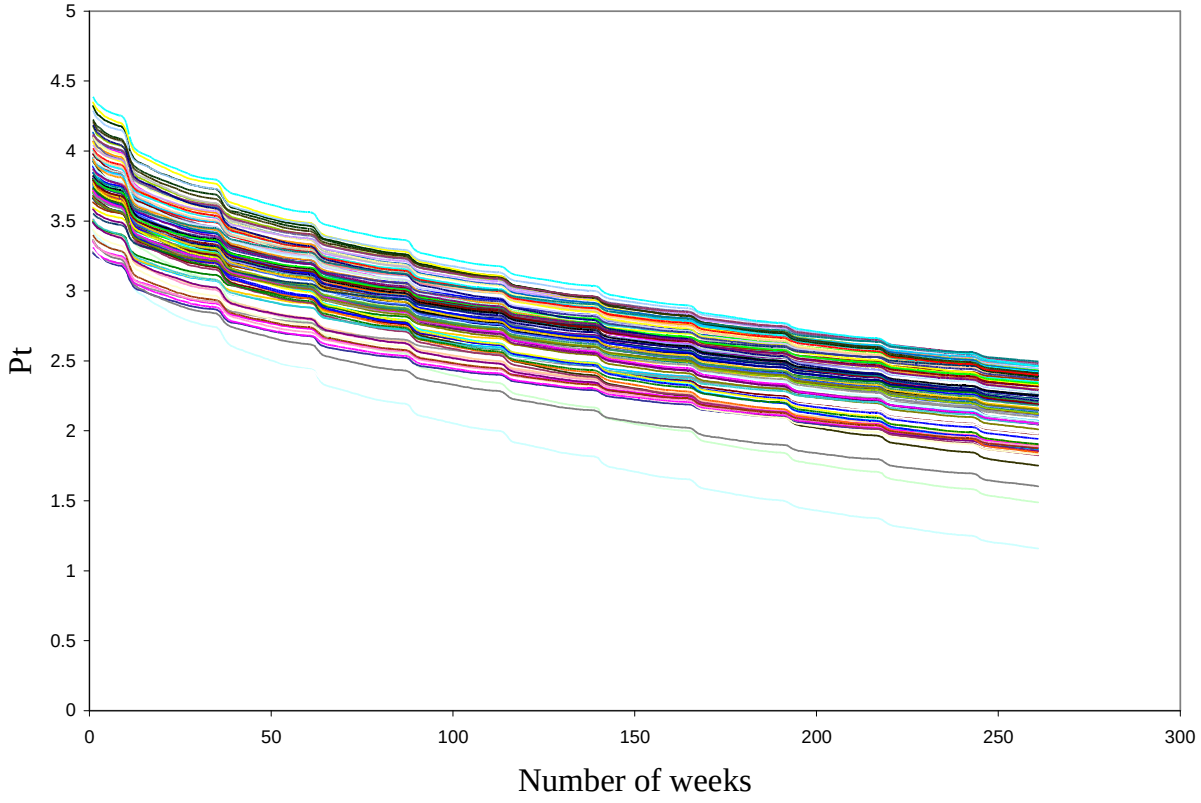

Figure 2 Performance variability of 100 pavement realizations over the design life (10 years) for 50% AASHTO design reliability.

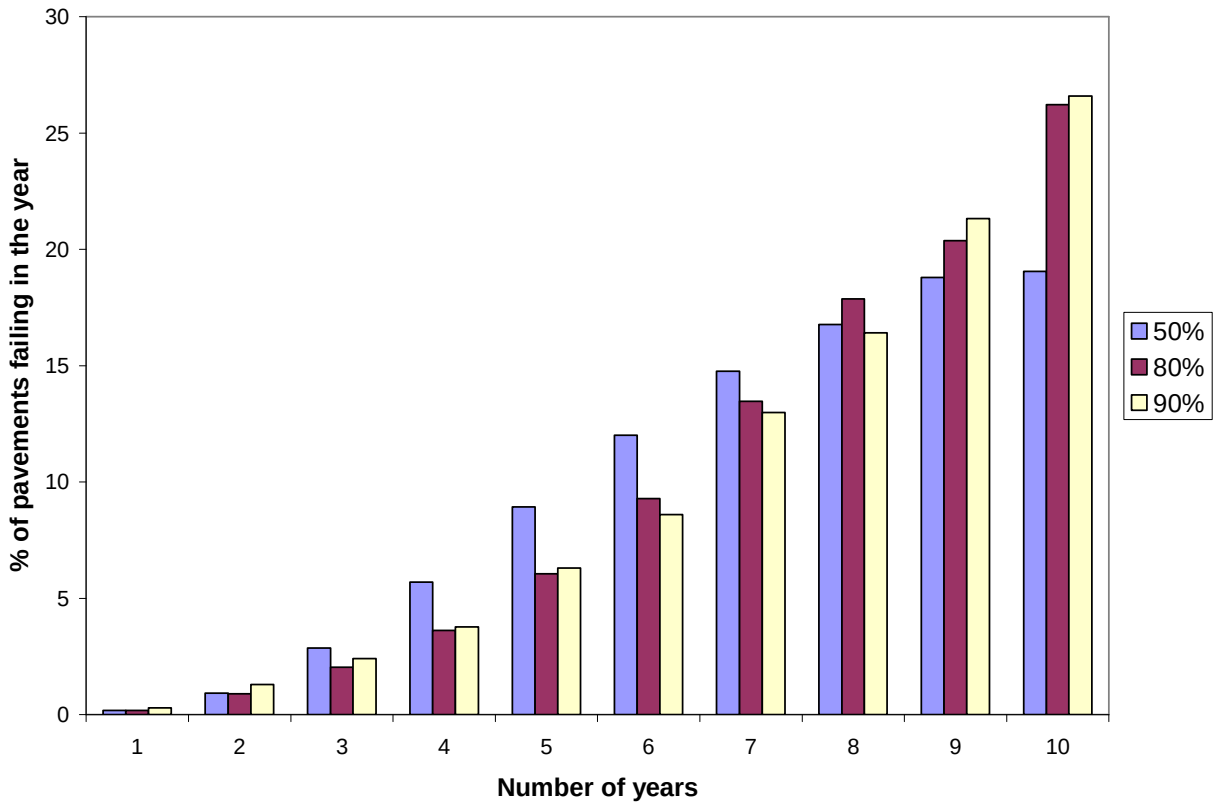

Figure 3 Percentage of pavements failing in a year during the design life

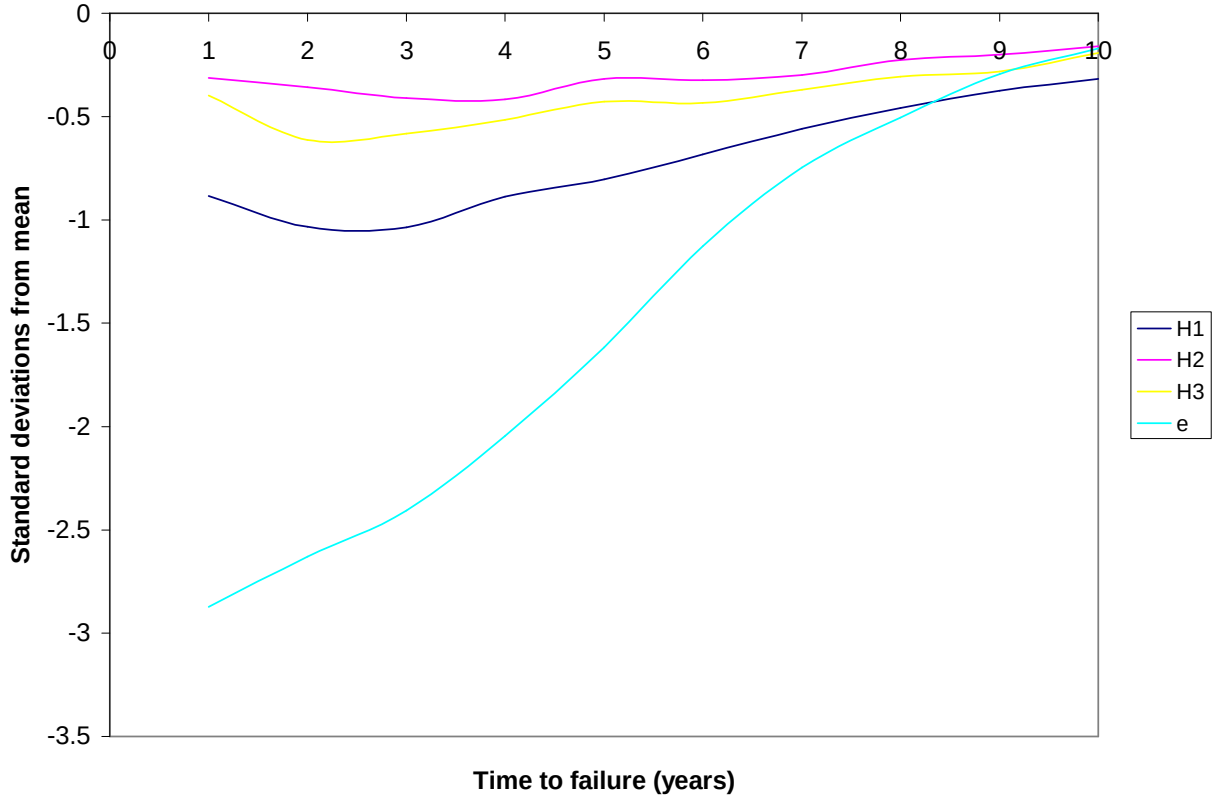

Figure 4 Importance of design variables (expressed by the average number of standard deviations from their mean values) as a function of time to failure.

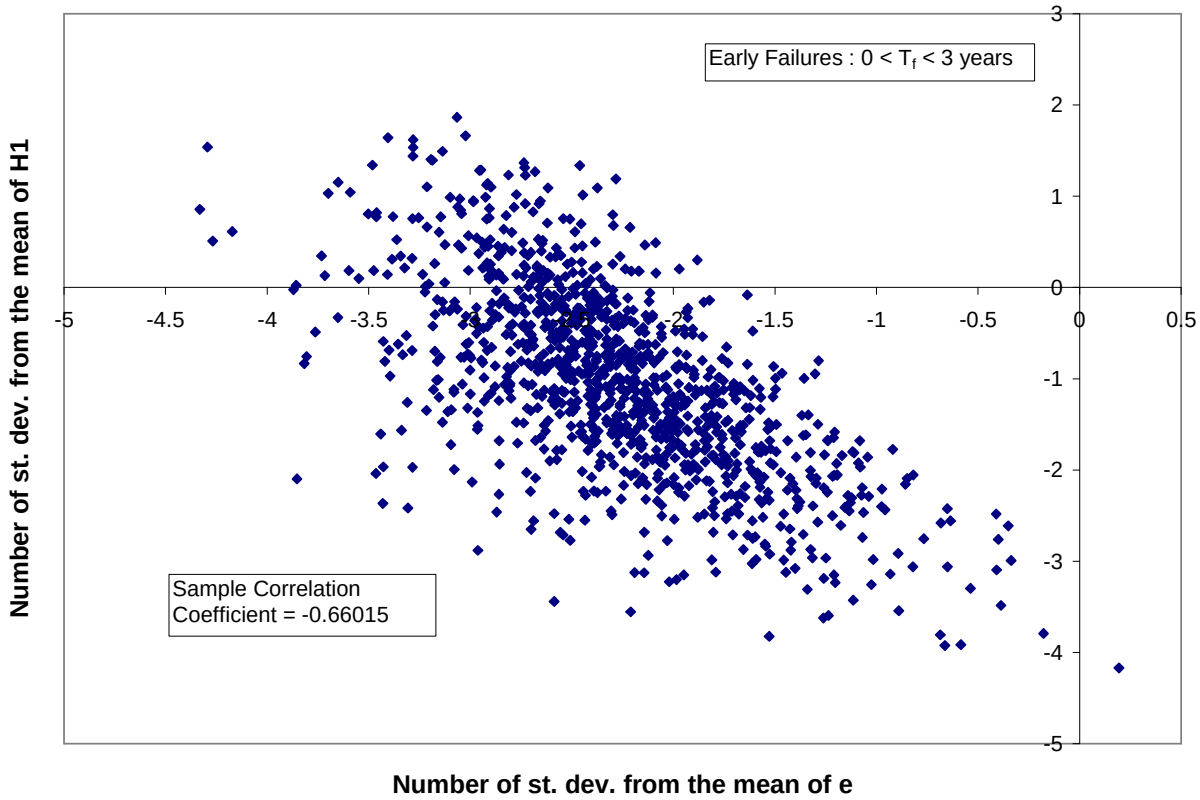

Figure 5 Variation of model error and surface thickness among pavements that experienced early failures.

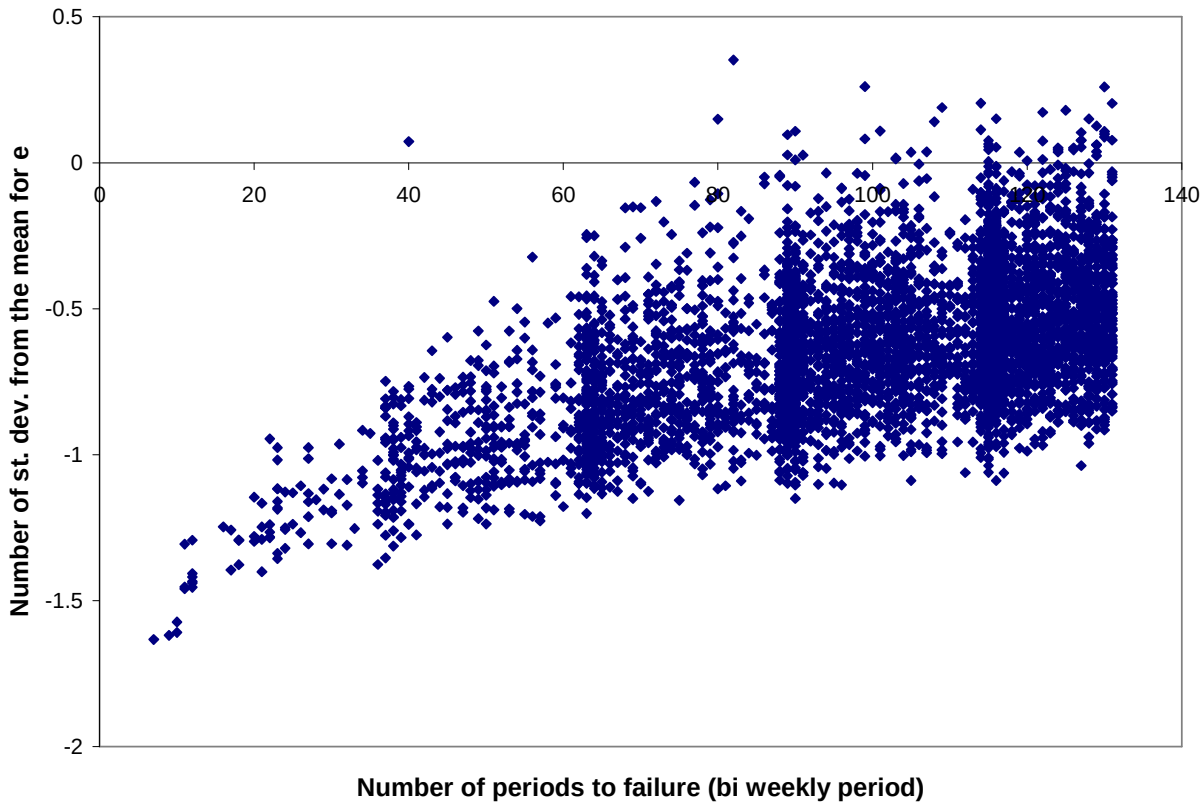

Figure 6 Variation of model error with time to failure for the failed pavements