Influence of Statistical Variation in Falling Weight Deflectometers on Pavement Analysis

RAJ SIDDHARTHAN, PETER E. SEBAALY, AND MOHAN JAVAREGOWDA

A relatively simple approach that is based on a Monte Carlo simulation procedure is presented to statistically investigate the influence of variation in falling weight deflectometer (FWD) measurements on pavement analysis. The factors considered are pavement moduli and pavement performance or response strains that correlate with pavement performance. From extensive FWD data obtained in Nevada, the variability of FWD deflections was statistically quantified at six sites for two seasons. The FWD tests were carried out at an interval of 50 ft within a uniform 1,000-ft test pavement section at every site. Using these data, as many as 900 FWD sensor deflections with normal distribution for each sensor were randomly generated for each of the 12 cases (six sites, two seasons) studied. The MODULUS program was used in the backcalculation of the pavement layer moduli. The FWD measurements show substantial variation within all of the uniform pavement sections investigated. Backcalculated pavement layer moduli using the generated deflection data show a large variation. The coefficient of variation for layer moduli vary from 5 to as much as 65 percent. Larger variations were computed for AC and base. However, the coefficient of variations for pavement performance strains are smaller, varying from 8 to 25 percent. Because the number of loading cycles to cause pavement distress is sensitive to pavement performance strains, the variation in pavement strain caused by the variation in FWD measurements has a significant influence on pavement life predictions.

As a consequence of the decreasing number of new highway construction projects, much attention is devoted to upgrading and maintaining existing highways. In this regard, effective and often customized overlay design procedures are being actively researched by state transportation departments. The primary objective of an overlay design analysis is whether a highway section requires overlaying and, if it does, by how much. If an overlay is needed, then the goal is to arrive at a pavement section that can withstand the expected traffic loads throughout the design life without excessive pavement failures such as cracking, rutting, or loss of serviceability.

The stiffness of the existing pavement layers is an important input parameter in the overlay design procedures. Although both laboratory and nondestructive testing (NDT) may be used to estimate the stiffness (modulus) of pavement layers, the NDT methods have become widely accepted because they are cheaper and more practical. Through the years, NDT methods have undergone changes because of the need to more realistically simulate wheel loading. The simulation of wheel loading has evolved from a static load to a more representative impulse loading using the falling weight deflectometer (FWD). The FWD is one of the most widely used NDT devices and is being used extensively in the Long-Term Pavement Performance (LTPP) evaluation that is being undertaken by the Strategic Highway Research Program (SHRP). The pavement layer moduli are computed using a "backcalculation procedure" in which the measured deflections on the pavement surface under FWD loading serve as input.

In a typical pavement improvement project, a stretch of highway is divided into a number of representative pavement sections with similar pavement and traffic loading characteristics. Among other factors, important parameters such as age, construction data, layer thicknesses, and pavement condition are often used to determine the extent of representative pavement sections. Within a representative pavement section, FWD tests are carried out at an interval that typically varies from 500 to 1,000 ft. In Nevada, the interval is often 500 ft. The pavement layer properties near an FWD test location are assumed to be uniform. A deviation from this assumption will affect backcalculated moduli and, thus, introduce error in the subsequent pavement analysis procedures that use FWD-based results.

A number of sources of error are associated with FWD measurements. The influence of all of the error sources cannot be completely eliminated. For instance, human factors can influence FWD testing but cannot be predicted. An additional factor to be considered is the pavement section variability. Even though the asphalt concrete (AC) and base layers may have relatively constant properties, the subgrade conditions often vary erratically even within a short distance.

The University of Nevada, Reno, and the Nevada Department of Transportation (NDOT) have collected an extensive FWD data base on a variety of pavements located around the state during at least 4 years and for all four seasons. This data base, in which the FWD measurements were taken at 50-ft intervals within a representative uniform pavement section, has been used to study statistically the variability in pavement deflections caused by various error sources. FWD data collected at these much finer intervals are considered to reflect the true variation of the material properties within a pavement section that is assumed to be uniform. Under these circumstances, the FWD deflection measurements collected at finer intervals can be considered as random variables, and a probabilistic approach seems appropriate to interpret the results given by subsequent backcalculation analysis. This paper uses a Monte Carlo simulation procedure to quantify the

---

Department of Civil Engineering (258), University of Nevada, Reno, Nev. 89557.

influence of error in FWD deflection measurements on pavement moduli. The data base of pavement moduli values generated by this approach can be used to study the corresponding layer moduli variation. One of the main purposes of estimating the pavement layer stiffnesses is to use them to compute the stresses and strains in the pavement under the design wheel loads. These stresses and strains can then be used as input to pavement distress (performance) models (e.g., rutting and fatigue). The investigation of the influence of the variation in FWD sensor deflections on pavement strains is also presented in this paper.

BACKGROUND

The FWD causes the pavement to deflect by dropping a free-falling mass that strikes a spring buffered plate. By changing the mass, the drop height, or both, the impulse force on the pavement can be changed. The deflections of the pavement surface are measured at a number of predetermined locations using velocity transducers. These deflection data, along with the thickness of pavement layers and other pavement properties, are used to predict the resilient modulus of the layers using a certain backcalculation procedure. By far, the most widely used backcalculation procedures assume static layered elastic conditions. The pavement materials are characterized to be elastic, homogeneous, and isotropic, with full contact at layer interfaces. The bottom boundary may be assumed to be located at some depth below the top of the subgrade or at a very large depth (half-space).

A recent NCHRP research program on the nondestructive evaluation of pavement layer stiffnesses has assessed all types of NDT equipment for both project-level and network-level pavement condition evaluation (*1*). This extensive study clearly outlines the applicability and limitations of FWD testing to obtain pavement layer stiffnesses. The research concluded that FWD testing is the most suitable for both project- and network-level pavement evaluation. The study also outlines a much more efficient backcalculation procedure that is based on linear elastic theory in which the best values of the layer moduli are estimated using interpolation between calculated deflection basins. This backcalculation analysis method is incorporated into a program called MODULUS. This paper uses the MODULUS program to backcalculate pavement layer moduli. Several state transportation agencies and private firms are currently using MODULUS.

Mechanistic or mechanistic-empirical design procedures characterize the long-term performance of pavements in terms of basic performance or response parameters such as stresses and strains that are induced in the pavement. In these methods, the failure is normally defined in terms of specific mechanisms such as fatigue cracking, rutting, and low temperature cracking. Mechanistic-empirical methods depend in part on empirical relationships between pavement stresses and strains and the number of load applications that the pavement can support before failure. For example, the strain at the bottom of the existing asphalt layer is normally correlated to fatigue failure. In some instances, the stress or strain at the top of the subgrade has been used to correlate rutting failure. In general, mechanistic-empirical procedures have been widely accepted by practitioners and researchers in recent years because they have been proven to be somewhat more reliable and because the pavement performance parameters can be obtained easily.

One of the major uses of the layer stiffnesses is to obtain the pavement performance parameters mentioned above. Static multilayer linear elastic computer programs, such as ELSYM5 and CHEVRON, are often used to estimate these pavement performance parameters. It has been argued that, if both the pavement performance parameter evaluation and FWD backcalculation procedures are carried out under a consistent set of conditions (i.e., static layered elastic), the dynamic effects and nonlinear soil properties may not be important.

It will be clear from the next section that there are a number of sources of error in FWD deflection measurements that cannot be totally eliminated. The paper investigates the influence of the variation in FWD measurements caused by such errors on the backcalculated pavement layer moduli and also on the pavement performance parameters mentioned earlier.

RESEARCH APPROACH

Sources of Variability in FWD Testing

It is important that the sources of variability associated with FWD testing be identified and reduced to a minimum. In general, there are two kinds of errors: random and systematic. These error sources result in a variation in the FWD measurements even within a uniform pavement section. Systematic errors can be identified and may be quantified, whereas random errors are a result of random variations in the measurements or in the pavement materials. Random errors are present in measurements recorded by the load cell and velocity transducers. Although they may be reduced somewhat by repeated testing and averaging, random errors cannot be totally eliminated. For example, Dynatest FWD deflection sensors have an accuracy of approximately ±2 percent as provided by the manufacturer. Other sources of random error include spatial variation of the material properties, both with depth and along the pavement length, and distortion in the deflection caused by passing traffic in the adjacent lanes.

On the other hand, systematic errors are introduced by bias; therefore, their effects may be eliminated by removing the source of the bias. Lytton et al. (*1*) have documented a number of systematic error sources. These sources include (a) erroneous assumptions made in the backcalculation process (e.g., the applicability of the static linear elastic approach), (b) a deviation of the contact pressure from a uniform distribution, and (c) a temporal variation in material properties within one layer caused by significant thermal and suction gradients.

Existing pavements are seldom perfectly flat; therefore, when the FWD falling mass strikes the base plate, uniform contact pressure may not be present under the plate. Furthermore, to achieve uniform contact pressure, the plate should be substantially flexible enough to deform and match the pavement surface. Recent studies by Touma et al. (*2*) and Uzan and Lytton (*3*) show that significant errors may result in the backcalculated layer moduli values if full contact between the base plate and pavement is not present.

Nevada FWD Data Base

NDOT is sponsoring a research project at the University of Nevada, Reno, to develop a customized overlay design procedure that takes into account the localized conditions that exist throughout the state. The overlay design procedure that is being developed uses a mechanistic-empirical approach in which pavement performance is predicted on the basis of stresses and strains induced in the pavement. As a part of this project, a total of 27 representative highway sites were selected for monitoring on the basis of factors such as climate, traffic, age, and type of construction. At each of the sites, the test section consists of a 1,000-ft highly uniform section on the outside traffic lane. The uniformity of the test sections was checked through coring and testing all of the layers in the pavement structure. Each test section was subdivided into 21 stations at 50-ft intervals, and FWD testing using Dynatest FWD model 8000 was performed for each of the four seasons for at least 2 years. Tests were carried out at four load levels varying from 6,000 lb to as much as 20,000 lb. The data base that was generated by this project is quite substantial.

Variation in FWD Deflection Data

The influence of the variation of the material properties can be somewhat reduced by performing FWD testing at finer intervals within a test section, but it cannot be totally eliminated. In this paper no attempt is made to quantify the influence of the individual error sources identified earlier. The influences of all of the error sources on the FWD measurements are lumped together, and the variability of the measurements within a uniform pavement section is statistically quantified as described later.

From the FWD data base described above, the deflection data collected for six sites that have a thin to medium-thick AC layer were selected for an in-depth study in this paper. The pavement section thicknesses at these sites and the dates of the FWD measurements are shown in Table 1. Only measurements that were obtained for two seasons in 1988 and 1989 (summer and winter) were considered in this paper. Typical deflection results obtained below the center of the plate (D-1) and at a point 55.1 in. from the plate (D-6) in August 1988 (summer) and February 1989 (winter) at Site 16 are shown in Figure 1. The deflections that correspond to the FWD load that is closest to 9,000 lb were selected and normalized to an FWD load of 9,000 lb.

The deflections can be seen to vary substantially within a test section. Among other reasons, one of the primary causes behind the variation in the deflection between the stations is the variability in the subgrade. This is because the other factors, such as the pavement cross section, construction method, and material types used at the site are the same. The winter and summer deflections are similar in shape; however, the summer values at D-1 (Figure 1 (*top*)) are higher, whereas the winter values at D-6 (Figure 1 (*bottom*)) are higher. This difference is because the deflection at D-1 is affected by the material properties of all the pavement layers, whereas the deflection at D-6 is affected by the properties of subgrade only. A histogram obtained for the deflection data at D-1 is presented in Figure 2. This histogram was obtained with an interval of 0.5 mils for August 1988. The histogram shows that the variation of deflection can, for practical purposes, be considered to be normally distributed. The mean, standard deviation, and coefficient of variation of the deflections of all the FWD sensors have been computed, and the values associated with the first five transducers are presented for all the sites in Table 2.

The mean deflection at Site 28 is substantially higher than that at other sites because this site has the lowest AC and base thicknesses. The coefficient of variation (COV) of deflections varies between 9.0 percent and as much as 40.8 percent. Generally, higher COV values occur for sensors located farther away from the center of the plate, where measured deflections are small.

Proposed Monte Carlo Simulation Procedure

It is clear from the previous discussion that the FWD deflections can vary substantially within a short distance and may be represented by a normal distribution. It was decided to use the Monte Carlo simulation approach to investigate the influence of the variation in the FWD deflection (*4,5*). The variables investigated in the study are the backcalculated pavement layer moduli and pavement performance parameters.

A random number generator routine that is available in a computer library generates random numbers with a uniform density; therefore it was not used in the study. Random num-

TABLE 1 Details of Selected Test Sites and Dates of FWD Measurement

Site No.	FWD Measured Dates	A.C. thickness (in)	Base thickness (in)
11	8-09-88 and 2-06-89	4.00	11.00
12	8-09-88 and 2-06-89	7.25	16.00
16	8-10-88 and 2-07-89	7.75	11.00
24	6-08-88 and 12-12-88	7.75	9.00
26	3-08-88 and 6-07-88	7.50	9.00
28	3-11-88 and 6-09-88	3.00	6.00

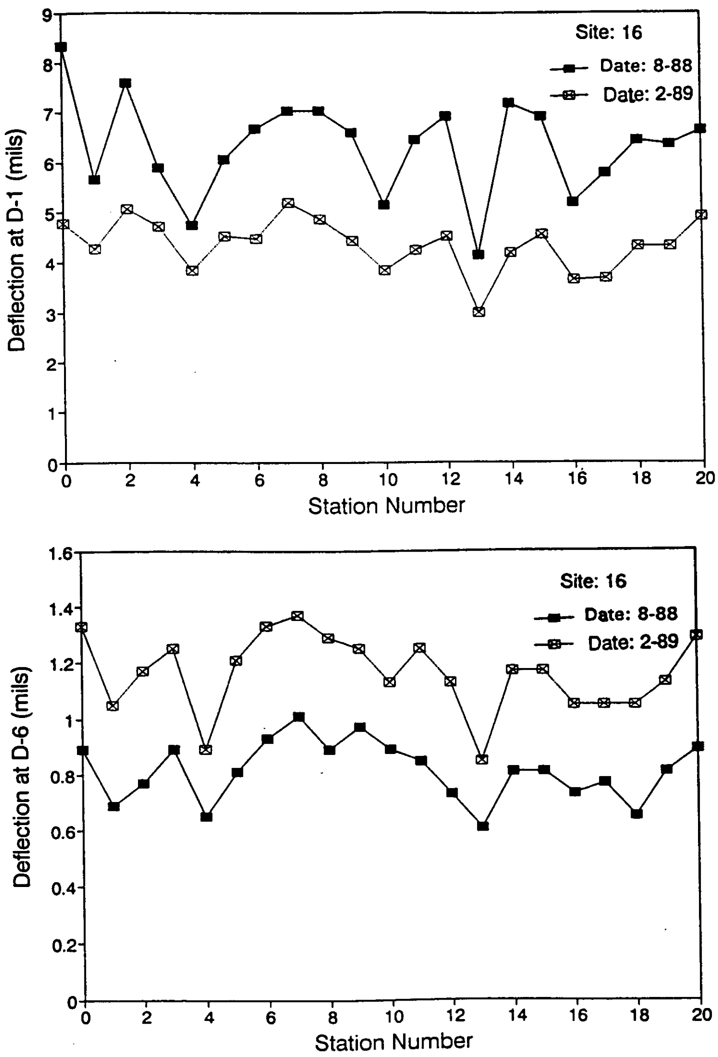

FIGURE 1 Variation of D-1 (*top*) and D-6 (*bottom*) across Site 16.

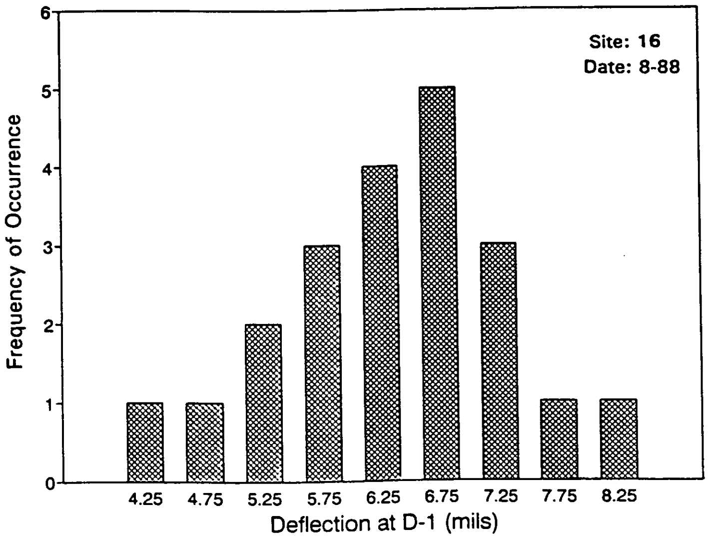

FIGURE 2 Histogram of D-1 across Site 16.

TABLE 2 Summary of Mean and Standard Deviation of Measured Deflection for Selected Sites

Site	Date	Description	D1	D2	D3	D4	D5
11	8-88	Mean	5.81	4.02	2.94	1.18	0.69
		Std.Dev/C.O.V(%)	0.67/11.5	0.59/14.7	0.51/17.2	0.26/22.3	0.13/19.5
	2-89	Mean	9.90	7.87	6.54	3.21	1.84
		Std.Dev/C.O.V(%)	1.80/18.0	1.76/22.4	1.74/26.6	1.17/36.4	0.73/39.5
12	8-88	Mean	4.79	3.86	3.25	1.81	1.09
		Std.Dev/C.O.V(%)	0.97/20.2	0.80/20.7	0.67/20.6	0.42/23.2	0.28/26.2
	2-89	Mean	4.33	3.60	3.10	1.84	1.21
		Std.Dev/C.O.V(%)	1.41/32.7	1.10/30.6	0.85/27.1	0.50/27.1	0.36/29.5
16	8-88	Mean	6.65	5.04	4.15	2.43	1.57
		Std.Dev/C.O.V(%)	0.99/14.8	0.78/15.5	0.67/16.1	0.42/17.3	0.26/16.5
	2-89	Mean	4.19	3.70	3.41	2.48	1.84
		Std.Dev/C.O.V(%)	0.52/12.4	0.46/12.5	0.43/12.6	0.34/13.5	0.24/13.4
24	6-88	Mean	13.74	11.89	10.55	6.62	4.06
		Std.Dev/C.O.V(%)	1.24/9.0	1.13/9.5	1.03/9.7	0.73/11.0	0.54/13.4
	12-88	Mean	9.25	8.30	7.61	5.38	3.67
		Std.Dev/C.O.V(%)	0.89/9.6	0.82/9.8	0.78/10.1	0.62/11.5	0.49/13.4
26	3-88	Mean	8.12	7.24	6.63	4.82	3.33
		Std.Dev/C.O.V(%)	1.27/15.6	0.91/12.6	0.82/12.4	0.54/11.2	0.36/10.9
	6-88	Mean	10.82	9.18	8.13	5.25	3.46
		Std.Dev/C.O.V(%)	1.58/14.6	1.18/12.9	1.04/12.8	0.78/14.8	0.36/10.5
28	3-88	Mean	50.87	38.18	27.64	11.35	6.85
		Std.Dev/C.O.V(%)	6.03/11.8	6.34/16.6	3.81/13.7	1.84/16.1	1.02/14.8
	6-88	Mean	51.26	38.60	28.23	11.42	6.80
		Std.Dev/C.O.V(%)	6.21/12.1	4.71/12.2	3.70/13.0	1.82/15.9	1.15/16.9

bers with a normal distribution can be obtained using the following equation (*6,7*):

$$X = \sqrt{-2 \log (U)} \cos(2\pi V) \qquad (1)$$

where U and V are two independent random variables with uniform densities on [0,1]. U and V can be generated using the standard routines available in the computers. The variation of X given by Equation 1 has a mean and a standard deviation of 0 and 1, respectively. Now the random variable Z having a normal distribution with the mean of $\overline{Z}$ and a standard deviation of σ can be obtained by using

$$Z = \overline{Z}X + \sigma \qquad (2)$$

This procedure was used to obtain all of the FWD sensor deflections independently for each site and each season using the means and standard deviations summarized in Table 2. In total, as many as 900 sets of deflection basins per case were generated for all 12 cases identified in Table 2. In each set, the sensor deflections were obtained independently of each other using Equations 1 and 2. Only the randomly generated deflections that fell within the mean ± standard deviation were used in the computations. Typical histograms obtained for Site 16 using the randomly generated deflections at D-1 and D-6 are shown in Figure 3. The variation indicated in Figure 3 (*top*) compares favorably with the measured variation in Figure 2; in addition, the mean of the deflections match with those given in Table 2.

Results of Backcalculation Procedure

The deflection basins obtained using the Monte Carlo simulation procedure were used with the MODULUS program to backcalculate layer moduli (*1*). The input data for the MODULUS program include layer thicknesses, Poisson's ratio, ν, for the layers, and estimated maximum and minimum resilient moduli values for the layers. These input parameters are shown in Figure 4. The sensor locations for D-1 through D-7 are located at 0, 7.9, 11.8, 23.6, 36.0, 55.1, and 70.9 in. from the center of the plate. The thickness of the subgrade layer is not required when using the MODULUS program because it is treated as an additional unknown variable. The program computes a series of deflection basins and tries to match the input deflection basin to arrive at the layer moduli

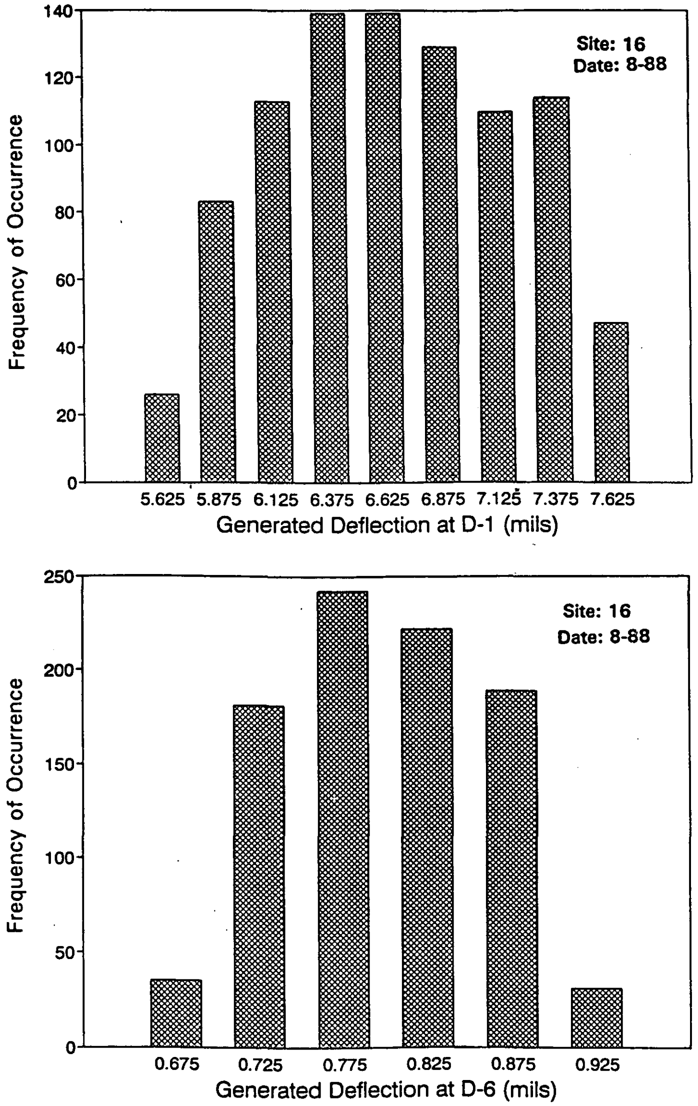

FIGURE 3 Variation of randomly generated deflection at D-1 (900 values) (*top*) and D-6 (900 values) (*bottom*).

values and the thickness of the subgrade. This program is much more efficient than other backcalculation programs and, in general, has been found to yield reasonable results.

Table 3 summarizes the backcalculated moduli values for all the sites and seasons considered in the study. The table gives results in terms of the mean, standard deviation, and coefficient of variation. Figure 5 shows the typical variation obtained for the AC resilient modulus at Site 16 (August 1988). These results clearly show that a substantial difference in the backcalculated moduli exists for the randomly generated deflection basins. In the case of AC and base, the coefficients of variation vary from 12 to as much as 65 percent. The coefficient of variation in the case of subgrade is smaller, varying between 5 and 13 percent. On the average, the mean AC modulus of thin pavement sections (Sites 11 and 28) is not affected by the seasonal variation. Site 16 showed the largest variation.

Pavement Performance Parameters

Pavement performance criteria such as AC fatigue and pavement rutting can be investigated for anticipated traffic loadings of existing pavements using layered linear elastic models, with layer moduli obtained from FWD data. A comprehensive review of AC fatigue failure studies indicated that crack initiation in AC is related to the maximum tensile strain, ε_a, in the AC layer (*8,9*). Some models suggest that the AC modulus and the ε_a are important factors that affect the AC fatigue life. However, the relative importance of the AC modulus is much less than that of the ε_a. The maximum tensile strain in medium-thick pavements occurs at the bottom of the AC layer.

The other major factor that affects pavement performance is surface rutting caused by permanent deformation in the pavement layers. Although the contribution of all the pave-

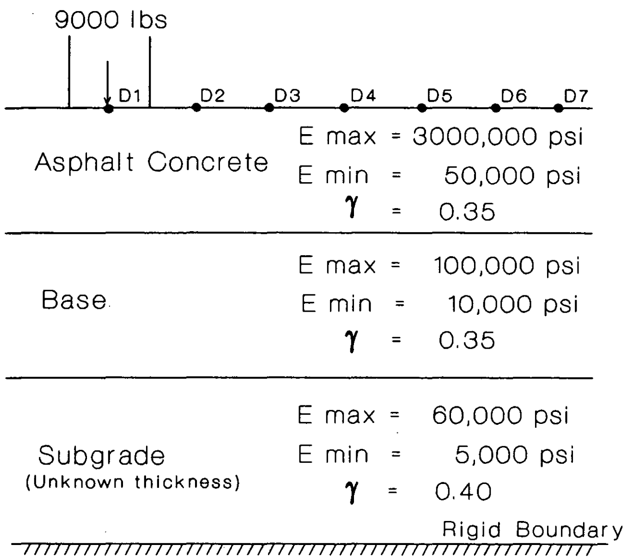

FIGURE 4 Input data for MODULUS program.

ment layers needs to be considered in rutting predictions, it is customary to pay the most attention to the contribution from the subgrade. This is because its contribution is frequently dominant. A number of widely used pavement rutting models use the maximum vertical compressive strain at the top of the subgrade, ε_s, as a measure of rutting in the subgrade (*9–11*). The design philosophy is that the pavement rutting can be reduced by controlling ε_s.

The backcalculated layer moduli values and the thickness of the subgrade given by the program MODULUS were input into the program NHELSYM5 to compute the pavement strains, ε_a and ε_s (*12*). This program, which is based on the well-known ELSYM5 program, is menu driven and user friendly. However, this program was further modified so that it can handle multiple data sets of layer properties. The single-axle dual-wheel configuration used with the modified NHELSYM5 program is shown in Figure 6. The tire pressures have substantially increased from 60 to 70 lb/in.^{2} in the 1960s to as much as 90 to 120 lb/in.^{2} in recent years (*11,13*). A value of 110 lb/in.^{2} for the tire pressure was used in the computations. The strains ε_a and ε_s (principal strains) at the bottom of the AC layer and at the top of the subgrade, respectively, were computed at three vertical sections, as shown in Figure 6. These vertical sections are located along the center line of one tire midway between the centers of the tires and at the edge of a tire. Only the largest value out of these three values of ε_a and ε_s was noted and is presented in Table 4. All 900 cases per site per season that were considered in the backcalculations were used in the computations. Similar to earlier results, the variation in terms of the mean, standard deviation, and coefficient of variation are presented for all of the sites. Figure 7 presents a typical variation of ε_a and ε_s for Site 16 (August 1988).

A number of observations can be made from Table 4 and Figure 7. First, it appears that, except for Site 28, the strains strongly depend on the seasonal variation. This is mainly because Site 28 does not experience large environmental changes between the seasons for which the FWD measurements were made. Second, even though a large variation in the backcalculated moduli values was obtained [coefficient of variation (COV) as much as 65 percent], the COVs of the computed strain values are much lower with a maximum of 25 percent. The range of COV for both strains is between 8 and 25 percent. This means that the pavement strain computation is not very sensitive to variations in the pavement layer moduli. Third, the COV for the AC strain, ε_a, is much higher than the COV of the subgrade strain, ε_s. Finally, the strains developed for Site 28 are substantially larger than those obtained for the other sites. This can be traced to the substantially lower AC and base thicknesses at Site 28.

TABLE 3 Backcalculated Resilient Moduli for Pavement Layers

Site	Date	Resilient Modulus of A.C (psi)			Resilient Modulus of Base (psi)			Resilient Mod. of Subgrade (psi)		
		Mean	Std. Dev	C.O.V (%)	Mean	Std. Dev	C.O.V (%)	Mean	Std. Dev	C.O.V (%)
11	8-09-88	1,292,138	407,769	31.55	88,498	14,214	16.06	45,658	5822	12.75
	2-06-89	1,056,659	683,608	64.69	68,067	24,056	35.34	18,629	2629	14.11
12	8-09-88	895,965	357,775	39.93	92,074	13,565	14.73	19,292	2053	10.64
	2-06-89	1,378,614	727,555	52.77	94,484	15,795	16.71	19,180	2814	14.20
16	8-10-88	478,178	149,928	31.20	91,295	13,994	15.32	23,476	1509	6.42
	2-07-89	2,719,507	330,678	12.15	89,335	19,659	22.00	21,708	1759	8.00
24	6-08-88	454,426	138,486	30.47	32,485	17,490	53.84	7,712	575	7.45
	12-12-88	880,663	268,202	30.45	71,615	30,558	42.66	9,174	770	8.39
26	3-08-88	1,208,384	492,612	40.76	67,714	34,042	50.27	11,451	647	5.65
	6-07-88	583,480	283,299	48.55	61,773	29,722	48.11	9,722	520	5.34
28	3-11-88	267,919	155,529	58.05	18,482	9,517	51.49	5,549	296	5.33
	6-09-88	294,666	147,909	50.19	15,975	6,751	42.25	5,432	340	6.25

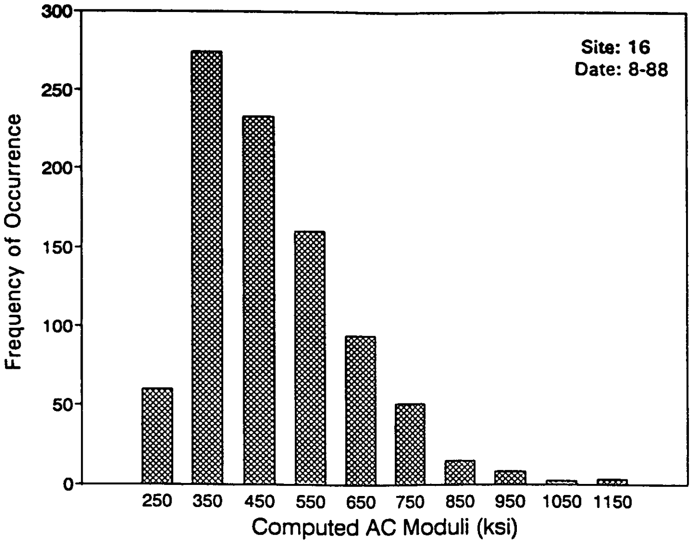

FIGURE 5 Variation of backcalculated AC resilient moduli.

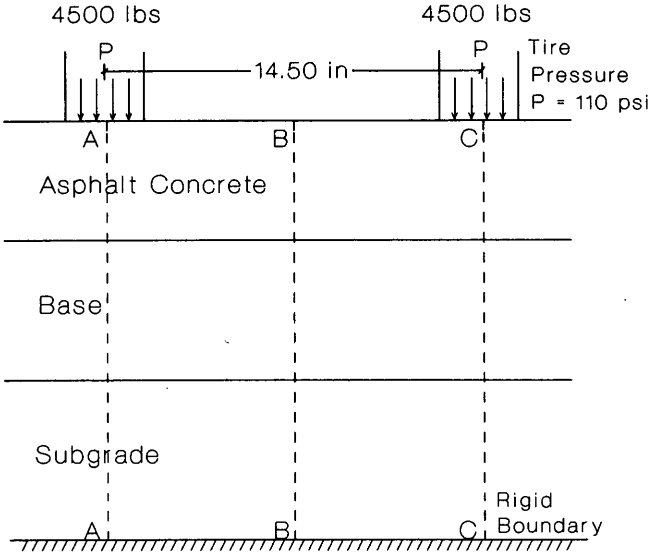

FIGURE 6 Single-axle dual-wheel configuration used for pavement strain computations.

Consequence of the Variability in Pavement Strains

A number of fatigue failure models are reported in the literature (*14–16*). One of the most widely used models is that of Monismith and Epps (*16*), which can be written as

$$\log N_f = 14.82 - 3.291 \log(\varepsilon_a) - 0.854 \log(E_{AC}) \quad (3)$$

where N_f is the number of cycles to fatigue failure (given in micro strain) and E_{AC} is the asphalt concrete resilient modulus (in ksi). This equation can be rewritten as

$$N_f = a(\varepsilon_a)^b \quad (4)$$

where a is a constant that depends on E_{AC} and $b = -3.291$. The equation can be used to compare the relative increase (or decrease) in the number of load applications for various strain values.

Using a representative mean value of 100μ and a variation of ±10 percent for ε_a, the changes in the number of cycles to fatigue failure can be compared using Equation 4. The calculations reveal that the ±10 percent change in ε_a corresponds to changes in the number of cycles of −27 and +41 percent, respectively.

Similar computations can be carried out for rutting using the Chevron equation (*10*). The Chevron equation gives the number of cycles, N_r, to cause a 0.75-in. rut depth as

$$N_r = 1.077 \times 10^{18} (\varepsilon_s)^{-4.483} \quad (5)$$

The ε_s should be given in terms of micro strain. Using a representative mean value of 200μ and a variation of ±10 percent for ε_s, the changes in the number of cycles to cause rutting failure can now be computed using Equation 5. A ±10 percent change in ε_s corresponds to changes in the number of loading cycles of −35 and +60 percent, respectively.

The changes in the number of loading cycles for both pavement failure models are substantial. This is because these models are sensitive to strain values.

SUMMARY AND CONCLUSIONS

The FWD testing, which is a widely used NDT method, simulates traffic loading more closely than other NDT methods. The surface deflections measured by FWD testing are used to estimate pavement layer moduli values using a "backcalculation procedure." The pavement moduli values are, in turn, often used with pavement performance models to predict the remaining life of pavement sections. There are a

TABLE 4 Pavement Performance Strains Computed Using Modified NHELSYM5

Site	Date	Max. Tensile Strain in AC (microns)			Max. Comp. Strain in Subgrade (microns)		
		Mean	Std. Dev	C.O.V(%)	Mean	Std. Dev	C.O.V(%)
11	8-09-88	98.6	13.5	13.6	- 145.2	12.0	8.3
	2-06-89	135.8	29.1	21.4	- 273.2	23.6	8.6
12	8-09-88	62.9	10.2	16.3	- 120.7	9.5	7.9
	2-06-89	51.2	12.6	24.6	- 108.3	14.3	13.2
16	8-10-88	79.7	7.6	9.6	- 158.5	8.8	5.6
	2-07-89	30.5	2.3	7.8	- 88.6	6.4	7.2
24	6-08-88	141.6	11.9	8.4	- 373.1	36.0	9.6
	12-12-88	74.3	6.4	8.6	- 232.7	21.3	9.1
26	3-08-88	65.1	8.4	12.9	- 196.4	28.6	14.5
	6-07-88	101.1	10.6	10.4	- 286.9	32.4	11.3
28	3-11-88	660.3	132.2	20.0	-1796.6	177.2	9.8
	6-09-88	682.4	116.5	17.0	-1805.0	187.7	10.4

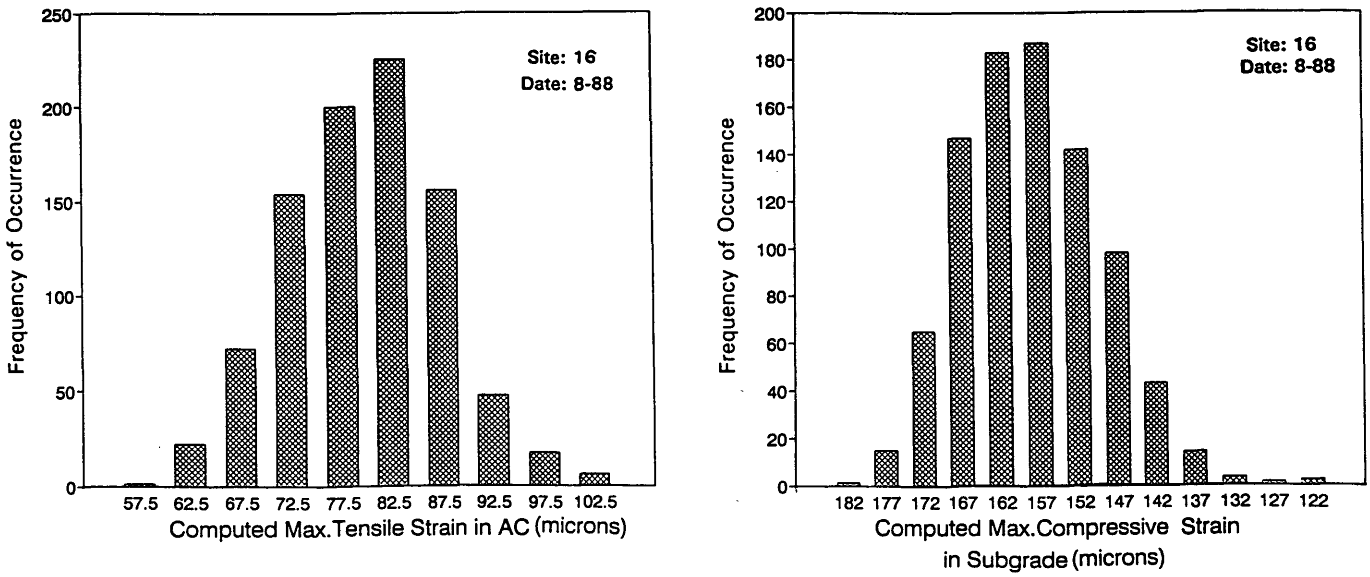

FIGURE 7 Variation of maximum tensile strain in AC (*left*); variation of maximum compressive strain in subgrade (*right*).

number of sources of error relative to FWD testing. These errors can be broadly divided into two types: systematic and random. Systematic errors include errors caused by assumptions made in the backcalculation process. Random errors are caused by, among other factors, deviations in sensor measurements, site variability, and passing traffic. Although they can be minimized by repeated testing and averaging, such errors cannot be totally eliminated. These error sources influence the FWD measurements taken even within a uniform pavement section. The study reported here investigates the influence of the variation in FWD deflection measurements on pavement moduli and pavement performance strain predictions.

As a part of the development of an overlay design procedure for Nevada, a total of 27 representative highway sites were selected and FWD tests were performed for each of the four seasons for at least 2 years. At each site, the test section consists of a 1,000-ft highly uniform pavement section, and FWD tests were carried out at 50-ft intervals. By selecting such fine testing intervals, the influence of the spatial variation in the material properties on the FWD measurements can be minimized. Deflection data from sites with thin to medium-thick pavements were selected to study the sensor deflection variability. The study reveals that the FWD measurements can be substantially influenced by the various error sources for all of the sites considered.

By varying the sensor deflections within its range, 900 sets of independent deflection basins using a random number generator with a normal distribution were obtained for the six selected sites and for two seasons. Backcalculation on these deflection basins was performed using the program MODULUS.

The data base of backcalculated moduli given by MODULUS for the generated deflection basins showed a substantial variation. In the case of AC and base, the coefficients of variation vary from 12 to as much as 65 percent. The coefficient of variation for subgrade is smaller, varying between 5 and 13 percent.

The pavement performance strains, such as the maximum tensile strain in the AC and the maximum compressive strain in the subgrade, were computed using a modified version of the computer program NHELSYM5. The backcalculated moduli and the subgrade thickness given by the program MODULUS were used as input. Even though the coefficient of variation was large for backcalculated moduli, the coefficients of variation of the computed strain were lower, with a maximum of 25 percent. The pavement performance strains were used to estimate the number of loading cycles required to cause pavement distress utilizing widely used pavement performance equations. This exercise suggests that the number of loading cycles to cause pavement distress is sensitive to pavement performance strains. This means that the variation in pavement strains caused by the variation in FWD measurements can have a substantial influence on the pavement life predictions.

Finally, the influence of other important factors such as the variation in the thickness of the AC and base have not been investigated in this paper. In the study, these thickness values, which were obtained from construction data and coring, were uniform across a test section. Studies investigating the influence of the changes in the thickness of the top layers may also be carried out using the proposed Monte Carlo approach.

ACKNOWLEDGMENT

The research reported here was funded by a grant from the Nevada Department of Transportation, whose support is gratefully acknowledged.

REFERENCES

1. R. L. Lytton, F. P. Germann, Y. J. Chou, and S. M. Stoffels. *NCHRP Report 327: Determining Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing*. TRB, National Research Council, Washington, D.C., June 1990
2. B. E. Touma, J. A. Crovetti, and M. Y. Shahin. The Effects of Various Load Distributions on the Backcalculated Moduli Values in Flexible Pavements. Presented at 70th Annual Meeting of the Transportation Research Board, Washington, D.C., Jan. 1991.
3. J. Uzan and R. L. Lytton. Analysis of Pressure Distribution Under Falling Weight Deflectometer Loading. *Journal of Transportation Engineering*, ASCE, Vol. 116, No. 2, 1990, pp. 246–250.
4. I. A. Shreider. *The Monte-Carlo Method: The Method of Statistical Trials*. Pergamon Press, New York, 1966.
5. J. M. Hammerslay and D. C. Handscomb. *Monte-Carlo Methods*. John Wiley Publications, New York, 1964.
6. L. J. Snell. *Introduction to Probability*. Random House/Birkhouser Mathematics Series, New York, 1988.
7. M. Dwass. *Probability: Theory and Applications*. W. A. Benjamin Inc., New York, 1970.
8. M. Gomez and M. R. Thompson. *Mechanistic Design Concepts for Full Depth Asphalt Concrete Pavements*. Transportation Engineering Series, No. 41. University of Illinois, Urbana-Champaign, Aug. 1984.
9. *Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1)*, 9th ed. Research Report 82-2. The Asphalt Institute, Aug. 1982.
10. L. E. Santucii. Thickness Design Procedure for Asphalt and Emulsified Asphalt Mixture. *Proc., 4th International Conference on the Structural Design of Asphalt Pavement*, University of Michigan, Ann Arbor, 1977, pp. 424–456.
11. O. K. Kim, C. A. Bell, and J. E. Wilson. Effect of Increased Truck Tire Pressure on Asphalt Concrete Pavement. *Journal of Transportation Engineering*, ASCE, Vol. 115, No. 4, July 1989, pp. 329–350.
12. R. Henry and C. Goodspeed. *NHELSYM5: Program for Pavement Analysis*. Department of Civil Engineering, University of New Hampshire, Oct. 1984.
13. D. R. Middleton, F. L. Roberts, and T. Chira-Chavala. Measurements and Analysis of Truck Tire Pressures in Texas Highways. In *Transportation Research Record 1070*, TRB, National Research Council, Washington, D.C., 1986, pp. 1–8.
14. A. A. Bu-Bushsait. *Development of a Flexible Pavement Fatigue Model for Washington State*. Ph.D. dissertation, University of Washington, Seattle, 1985.
15. P. S. Pell. *Special Report 140: Characterization of Fatigue Behavior: Structural Design of AC Pavements to Pavement Fatigue Cracking*. HRB, National Research Council, Washington, D.C., Jan. 1973.
16. C. C. Monismith and J. A. Epps. *Asphalt Mixture Behavior in Repeated Flexure*. Report TE-69-6. Institute of Transportation and Traffic Engineering, Berkeley, Calif., 1969.