3.0 NETWORK-LEVEL FWD TESTING

3.1 Introduction

Structural evaluation provides a wealth of information concerning the expected behavior of pavements *(Haas et al. 1994)*. However, due to the expense of data collection and analysis, structural capacity is not currently evaluated at the network level of pavement management by many agencies. The practice is more common at the project level of management. It has been argued that the structural capacity information, even derived from less intensive sampling than for project level purposes, can be very useful at the network work level for project prioritization purposes. The practice exists in a few states and Canadian provinces, such as Idaho, Minnesota, Utah, Alberta, and Prince Edward Island *(Haas et al. 1994)*. As mentioned earlier, due to limited resources and the large size of the network, network-level structural data collection annually in Kansas at the same rate (5 to 10 tests per mile) as the project level is not realistic. One of the objectives of this research was to determine the sample size (percent mileage), test intervals and frequency to be used as guides by KDOT for network-level FWD testing so that the deflection data can be used as input into the PSE computation process.

3.2 Data Collection

Deflection data was collected on the asphalt pavements in District IV from 1993 to 1996. KDOT maintains two types of flexible pavements - Full-Design and Partial-Design Bituminous Pavements. Full-Design Bituminous (FDBIT) pavements were designed for the current and projected traffic and usually carry heavier traffic than the Partial-Design Bituminous (PDBIT) pavements which resulted from the paving and maintenance of the original “farm to market” roads in the forties and fifties. District IV was chosen as the test network since its mileage most closely approximates the pavement types on the whole KDOT network and thus, deflection data collected on this district would be very representative of the KDOT network. The FDBIT and PDBIT pavement mileages in District IV are 545 and 695 miles, respectively. They represent roughly 15% and 14%, respectively, of the total network mileage in Kansas for the two pavement types. Data for this study was collected on the non-Interstate routes in District IV.

Pavement surface deflections were measured by a Dynatest 8000 Falling Weight Deflectometer (FWD). Ten (10) FWD tests per mile were performed on the outer wheel path of the travel lane. Table 3.1 summarizes the project details for data collection. FWD tests were conducted each year of the study period on the projects selected by NOS for the long-term rehabilitation program.. Thus the projects tested in a given year are the candidates for rehabilitation for a certain future year and should be in a "similar" condition state. The condition states are defined by NOS based on roughness, rutting, transverse cracking, fatigue cracking and/or block cracking. In total, approximately 20% of the FDBIT pavements and 36% of the PDBIT pavements from 96 "control" sections in District IV were included in the study.

Table 3.2 shows some geometric and loading characteristics of the sections selected. The annual ESAL's varied from 42,000 to 264,000 and are fairly representative of the traffic loads on KDOT's non-Interstate network. On average, the loading on the FDBIT pavements was three to four times the loading on the PDBIT pavements.

Table 3.1 Data Collection Summary

Year	Pavement Type				No. of Control Sections
	Full Design		Partial Design		
	Miles	% of	Miles	% of	
1993	36	6.6	107	15.4	43
1994	15	2.7	71	10.2	25
1995	25	4.6	9	1.3	11
1996	34	6.2	60	8.6	17
Total	110	20.1	247	35.5	96

Table 3.2 Characteristics of the Study Sections

Year	Pavement Type	Average Length (mile)	Average Annual ESALs	No. of Control Sections
1993	FDBIT	3.027	198,000	12
	PDBIT	3.359	71,000	31
1994	FDBIT	3.003	264,000	5
	PDBIT	3.548	58,000	20
1995	FDBIT	3.116	128,000	8
	PDBIT	2.686	44,000	3
1996	FDBIT	5.654	188,000	6
	PDBIT	6.624	42,000	15

3.3 Response Variables and Analysis Method

The following attributes were selected as response variables:

1. Normalized and Temperature-corrected first sensor deflection (d_1),

2. Subgrade Resilient Modulus (M_r), backcalculated from the FWD data following the

AASHTO Guide algorithm, and

3. Effective Pavement Modulus (E_p), also computed following the AASHTO Guide algorithm.

The FWD first sensor deflection values were normalized to 40 kN (9,000 lb) load level and then corrected to a temperature of 20° C (68° F) following the methodology proposed by Southgate and Deen and adopted by AASHTO *(AASHTO Guide 1993).*

3.4 Trends of Response Variables

Table 3.3 shows the summary statistics for d_1, M_r, and E_p for the years 1993 thru 1996 for the control sections. It appears that the coefficients of the variations for the backcalculated subgrade moduli were similar over the years, indicating the effects of spatial variation rather than variation over the time period considered. The coefficients of the variations are the highest for the E_p's which is derived from the other two parameters. It appears that the variabilities in those parameters are magnified in the calculation process. Table 3.3 shows the results of the student's t-tests between the means of these variables for the four years of study period. For all variables, there were no significant differences among the means of these variables for 1993, 1994, and 1995. Thus, the mean values of d_1, M_r, and E_p did not change significantly over three years. However, significant differences were noted between the first-sensor deflection values for 1996 and 1993 for both pavement types.

These results imply that the average structural capacity of the pavement network in Kansas most likely change over a three year period. In other words, it takes about three years of traffic and climatic affect to significantly change the average structural condition of the network.

3.5 Limit of Accuracy Curves

It is well known that tests conducted on pavement analysis units provide an estimate of the actual mean and standard deviation of the attribute under investigation. As the number of test

Table 3.3 Summary Statistics of the Response Variables

Variable	Year	Pavement Type							
		Full Design				Partial Design			
		Mean	Std. Dev.	C.V. (%)	n	Mean	Std. Dev.	C.V. (%)	n
d_1 (mils)	1993	11.3	5.6	50	12	23.6	10.3	44	31
	1994	9.6	0.8	9	5	24.3	10.5	43	20
	1995	14	5	36	8	19.7	5.5	28	3
	1996	19.3	9	47	6	19.7	7.2	37	11
M_r (ksi)	1993	17.7	4.3	25	12	12.5	3.3	26	31
	1994	14.9	3.1	21	5	10.7	3.1	29	20
	1995	16.4	4.2	26	8	13.2	2.6	20	3
	1996	12.7	3.2	25	6	12.6	2.0	16	11
E_P (ksi)	1993	250	190	75	12	318	241	76	31
	1994	267	110	40	5	447	412	92	20
	1995	149	58	39	8	352	167	48	3
	1996	207	115	56	6	317	285	90	11

Note: 1 psi = 6.89 kPa
1 mil = 0.025 mm

Table 3.4 Students t-test Results at 5% level of Significance

Response Variable	Pavement Type	Test	t-statistic	d.o.f.	Results
d_1	FDBIT	1996 vs. 1995	-1.413	7*	not significant
		1996 vs. 1994	-2.207	8*	not significant
		1996 vs. 1993	-2.309	16	significant
	PDBIT	1996 vs. 1995	-0.0076	12	not significant
		1996 vs. 1994	1.284	29	not significant
		1996 vs. 1993	2.141	40	significant
M_r	FDBIT	1996 vs. 1995	1.824	12	not significant
		1996 vs. 1994	1.183	9	not significant
		1996 vs. 1993	2.499	16	significant
	PDBIT	1996 vs. 1995	0.45	12	not significant
		1996 vs. 1994	-1.794	29	not significant
		1996 vs. 1993	0.059	31*	not significant
E_p	FDBIT	1996 vs. 1995	-1.118	7*	not significant
		1996 vs. 1994	0.902	9	not significant
		1996 vs. 1993	2.596	15*	significant
	PDBIT	1996 vs. 1995	0.199	12	not significant
		1996 vs. 1994	0.928	29	not significant
		1996 vs. 1993	2.287	34**	not significant

* unequal variances
** a few projects were eliminated due to unreliable thickness data

increases, the estimated value more closely approximates the true value. However, as mentioned earlier, more tests translate to more expenses and in some cases, unrealistic data collection and analysis expenses. The principles of statistical confidence levels can be used to determine how many tests will be necessary to ensure that the estimated mean is within a certain limit of the actual mean. Statistical limit of the accuracy curves helps assess the impact of the number of tests conducted on the precision of the estimate. The limit of accuracy, R, represents the probable range of the variation of the "true" mean from the average obtained by "n" tests at a given degree of confidence. Mathematically,

$$R = K \ (\quad / \ n) \qquad (3.1)$$

where,	K	=	standardized normal deviate, which is a function of the desired confidence level,
		=	standard deviation of the variable (d_1),
	n	=	number of FWD tests conducted or percent network mileage tested at a fixed interval, and
	R	=	allowable error in the random variable being considered.

It is to be noted that for a given confidence interval, standard deviation and number of tests, the corresponding error could be computed using Equation 3.1. For a given variable (e.g., deflection), if the confidence level (e.g., 95%), K and are known, the R value would be inversely proportional to the square root of the number of tests randomly selected. The relationship between the R value and the number of tests is depicted in Figure 3.1. AASHTO defines three zones along the accuracy curve. In Zone I, characterized by a steep slope, the precision of the estimate significantly increases with each additional test or sample and the benefit-cost ratios for increasing the number of tests per analysis are quite high. Zone III, on the other hand, is a region with little slope, where even large increases in the number of tests/samples obtained will not significantly improve the precision

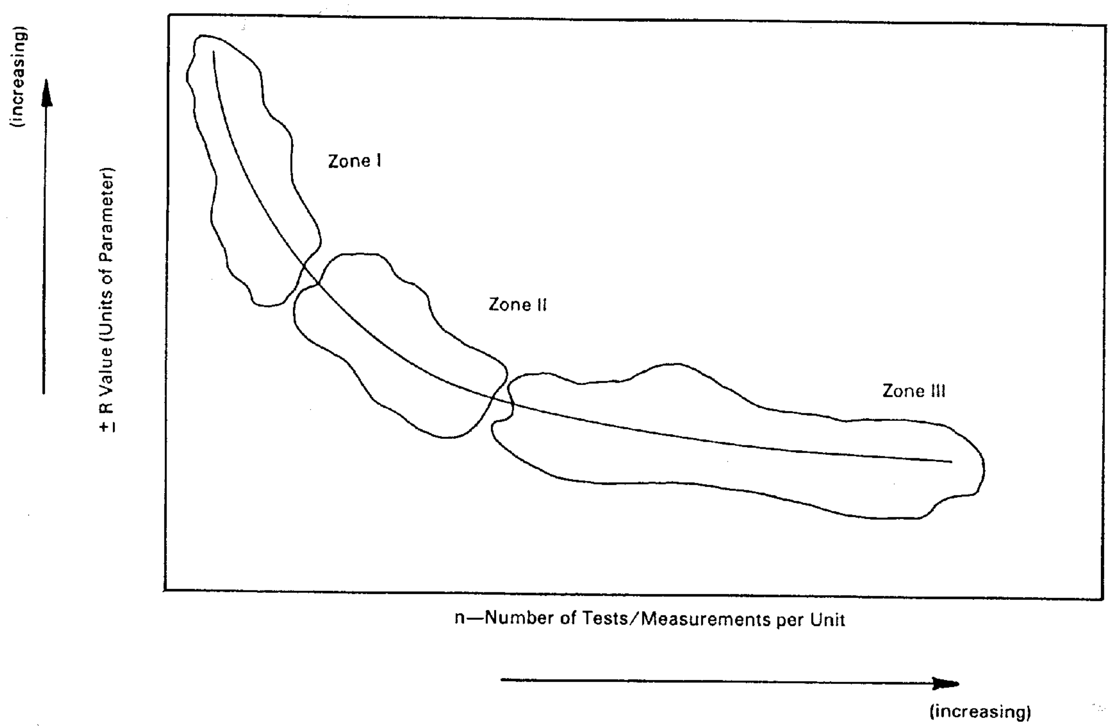

Figure 3.1 Typical Limit of Accuracy Curve for All Pavement Variables (*after AASHTO 1993)*

of the estimate, and the costs associated with additional testing may outweigh the benefits. Zone II represents the "optimal" range in developing a test program, because it represents the area where accurate estimates will be made using a minimum number of tests *(AASHTO Guide 1993)*.

3.6 Error Analysis

For this analysis, the temperature-corrected first sensor deflection (d_1) was chosen as the response variable and the values of d_1 for 1993, 1994 and 1995 were aggregated for the analysis. The error values associated with d_1 were computed as:

$$\% \text{ Error} = (\text{ Absolute Error/ Average value }) * 100 \qquad (3.2)$$

All error calculations were done at 95% confidence level for which the value of K is 1.96.

For each project, the average and standard deviation of the first-sensor deflections were computed. For error analysis of the FWD tests on the percentage of network mileage covered, it was assumed that the "true" standard deviation of the first-sensor deflections of each project is equal to the standard deviation obtained from the tests on 100% of the network covered without errors.

Table 3.4 shows the error analysis results for the network mileage tested. It is interesting to note that the percent error values corresponding to the percent network mileage tested are similar for the FDBIT and PDBIT pavements. Thus the percent error values for the two pavement types were combined and the following regression equation for the percent error was developed:

$$\text{percent } (\%) \text{ error} = \exp (4.096 - 0.5115 \ln (\% \text{ network mileage})) \qquad (3.3)$$

$$(R^2 = 0.976, \text{ Standard Error} = 1.142)$$

Figure 3.2 shows a plot of Equation 3.3. It is apparent that the FWD tests on more than approximately 20 percent of network mileage will not significantly increase the precision of the estimate or the first-sensor deflection value. Hence 20 percent mileage could be selected as a reasonable sample size in network-level structural evaluation of flexible pavements. This would

Table 3.5 Error Analysis Results

Pavement Type					
Full Design			Partial Design		
% Network	R	Error (%)	% Network	R	Error (%)
14	1.9	16	27	2.7	11
10.5	2.3	19	20	2.9	13
7	2.55	22	13.5	3.2	16
3.5	3.4	33	7	3.7	20

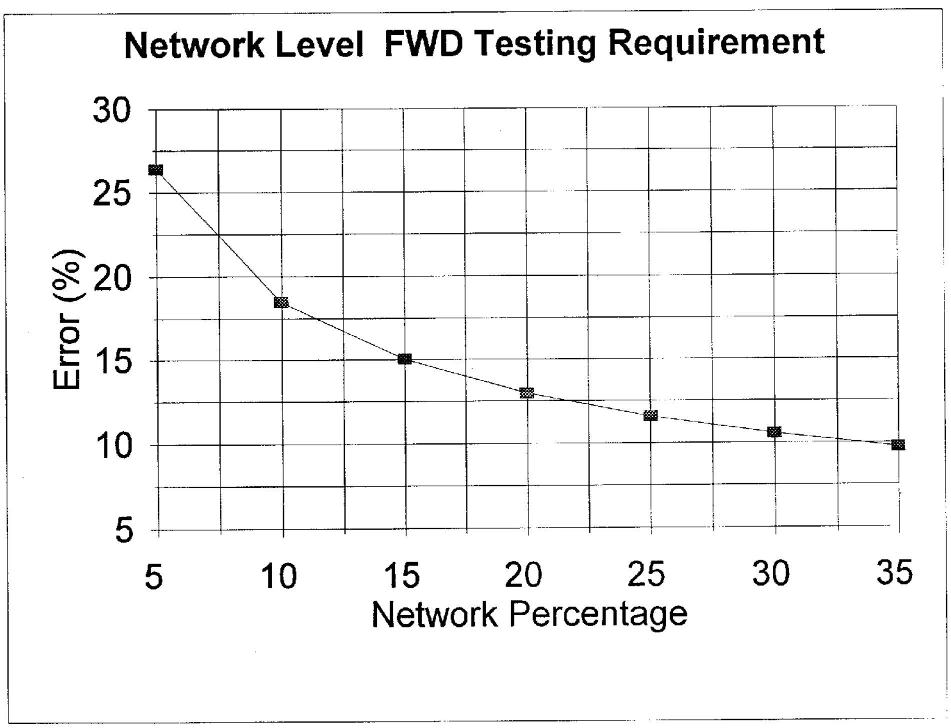

Figure 3.2 Network Level FWD Testing Requirements

translate into approximately 3,542 lane-km (2,200 lane-miles) of testing in three years. Thus, KDOT should test its system on a 3-year cycle or approximately 1,208 lane-km (750 lane-miles) each year for network evaluation. With two FWD units, this would require 19, 10-hour work days of testing each year.

For the error analysis of the FWD test rate on a particular project, it was assumed that the "true" standard deviation of the first-sensor deflections of each project is equal to the standard deviation obtained from 10 tests per mile. Percentage errors for the test intervals of seven, five, three, and one test per mile were computed. The 10 tests were done at about 160 m intervals. For seven tests per mile, every third test point was ignored. For five tests per mile, every other test point was ignored. For three tests per mile, the first, fourth and seventh test points were taken for analysis. The one test per mile was assumed to be at the beginning of each project. Results in Table 3.5 show that the average error does not vary significantly for seven, five, or three tests per mile. Thus, the lowest test rate, three tests per mile could be taken as the spatial test frequency at the network level.

The suggested test coverage of 20% mileage and spatial frequency of three tests per mile were tested with the FWD data collected in 1995. That year, 25 miles of FDBIT pavements were tested. Twenty percent mileage translated to only five miles of testing in 1995. Different combinations of the control sections which would result in five miles of testing showed that the average error for the spatial frequency of three tests per mile ranged from 14% to 16%, compared to 13% to 15% for five tests per mile, and 12% to 13% for seven tests per mile.

This testing would be necessary for network level structural evaluation of the KDOT pavements and also for using/updating the models to be developed in this study.

Table 3.6 Determination of the Number of Tests Per Mile at the Network Level

Percent error in FWD 1st sensor deflection for various test intervals (1995 data)				
Route	**Number of Tests Per Mile**			
	7	**5**	**3**	**1**
US 54	14	16	18	39
US 59	6	8	9	15
US 59	12	14	17	35
US 59	8	9	13	25
K 68	15	18	21	44
K 68	10	12	21	44
K 68	14	16	19	40
K 103	9	10	12	25
K 103	7	9	11	22
K 126	16	21	23	47
US 169	9	10	12	25
Average	11	12	14	29

3.7 Prediction of the Decrease in Structural Number

In this study, the network-level structural deterioration was predicted through quantification of the decrease in the structural number of the existing pavements estimated from the FWD data. This was necessary because this decrease in structural number will be used as a predictor for estimating PSE values for the control section. It is apparent that in the future, FWD test results will not be available for all control sections on the network. However, the decrease in structural number still could be estimated for any section based on the models to be developed.

The approach for structural evaluation was based on the second technique for pavement structural evaluation suggested by the 1993 AASHTO Pavement Design Guide. The technique, based on nondestructive testing (NDT) as discussed in Chapter 2 of this report, was used. Following this approach, the effective structural numbers (SN_{eff}) of the pavement sections were calculated using FWD data collected in 1993, 1994, and 1995.

The FWD first sensor deflection values were normalized to 40 kN (9,000 lb) load and were also corrected for temperature at 20° C (68° F). The deflection values were then used to calculate the subgrade resilient modulus (M_r). The effective E_p values were determined from Equation (2.4). Once the E_p value had been calculated, the effective structural number was found by the following formula provided by AASHTO:

$$SN_{eff} = 0.0045 * D * (E_p)^{1/3} \qquad (3.4)$$

The original structural numbers of the existing flexible pavements after rehabilitation actions, calculated according to the algorithms in KDOT's HYNELIFE program, were obtained from the KDOT's CANSYS database.

The decrease in structural number (SN) was then computed as:

$$SN = SN\ (CANSYS) - SN_{eff} \qquad (3.5)$$

3.7.1 Model Development

The major factors contributing to the structural deterioration of asphalt pavements are traffic and climate. In this study, the age of the pavement was taken as a surrogate variable for the climatic affect or aging. Three variables were selected to predict the decrease in structural number (SN) to assess structural deterioration at the network level:

1. Age (in years) of the pavement *since the last rehabilitation action*,
2. Cumulative number of ESAL's that have passed over the pavement *since the last rehabilitation action*, and
3. Thickness (in inches) of the *asphalt concrete (AC) layer*.

The thickness and rehabilitation histories of the pavement sections under study were collected from the HYNERES database of KDOT. Specifically, the following information was obtained:

(i) Years corresponding to different rehabilitation actions,

(ii) Type of rehabilitation action, and

(iii) Thickness of the overlay (s).

The AC layer thickness, the total thickness of the pavement sections above subgrade, and the age of the pavement since the last rehabilitation action were then calculated. The total thickness of the pavement sections is necessary during computation of the effective pavement modulus, E_p.

During this analysis, the FDBIT and PDBIT pavements were treated separately since the structural behavior of these pavements is different. By doing simple linear regression analysis, it was apparent that the decrease in structural number was highly correlated with the age, cumulative number of ESAL's and AC layer thickness for the FDBIT pavements, and the age and cumulative ESAL's for the PDBIT pavements. To select the correct variables, three variable selection methods of the Statistical Analysis System (SAS) software were used:

a. Forward Selection Method,
b. Backward Elimination Method, and
c. Stepwise Method

The results of these three variable selection methods are shown in Table 3.6. All three variables were selected for the FDBIT pavements, but the AC layer thickness was not selected for the PDBIT pavements. As mentioned earlier, PDBIT pavements are “built up” pavements-basically asphalt surfaced pavements which trace back to “farm to market roads” in the mid forties and fifties. The thicknesses of such pavements were really not designed to carry a specific traffic. This fact also is supported by the three independent variable selection methods of SAS indicating that the AC layer thickness of the existing pavement does not play an important role in determining the decrease in structural number of the PDBIT pavements. Therefore, thickness was dropped from the PDBIT model as a predictor variable. Also, a correlation study among the proposed variables revealed that the age and cumulative ESAL's are highly correlated to each other (64.3% for FDBIT and 62.1% for PDBIT pavements). Thus, to avoid multicolinearity, only one of them was included in the model, and the variable 'age' was selected because of its greater contribution to the R^2 value. Two types of models were selected in each case. The first one was a regular regression model with an intercept. The other model was forced to have a zero intercept. From a practical point of view, a zero-intercept model is more justifiable since it implies that the structural number will remain unchanged if the age since the last action is zero *(i.e., just after the rehabilitation action)* and the AC layer thickness is zero. For FDBIT pavements, the R^2 value for the intercept model was 83.4% and for the zero-intercept model, 81.3%. These values for the PDBIT pavements were 75.8% and 72.0%, respectively. For both types of pavements, the zero-intercept model was selected for being practical.

Table 3.7 Variable Selection Process Summary

Method of Selection	Variables selected by SAS	
	FDBIT Pavements	**PDBIT Pavements**
Forward Selection	1. Age 2. AC layer thickness 3. Cumulative ESAL	1. Age 2. Cumulative ESAL
Backward Elimination	1. Age 2. Cumulative ESAL 3. AC layer thickness	1. Age 2. Cumulative ESAL
Stepwise Method	1. Age 2. AC layer thickness 3. Cumulative ESAL	1. Age 2. Cumulative ESAL

3.8 Models Obtained and the 'Model Utility' Test

FDBIT Pavements: For the FDBIT pavements, the model to predict a *decrease* in structural number is:

$$SN = 0.0218 * age + 0.001 * AC\ layer\ thickness \qquad (3.6)$$

As shown in Table 3.7, the R^2 of the FDBIT pavements model is 0.8127. The significance values (p-values) for the parameters are: age: 0.0001 and AC layer thickness: 0.0176, indicating that both variables are significant at a level of more than 98%. The analysis of variance (ANOVA) for this model showed that the model has an F-value of 320 and its significance value is 0.0001. Since the selected model has a high F-value and a very low p-value, it satisfactorily passes the model utility test. The test shows that the model is helpful and adequate in predicting the dependent variable, SN. Also, the estimated root mean square error ( ) value for the model is 0.044, which indicates the selected model will predict the decrease in structural number (SN) at the network level with a variability of ±2 or ±0.088 for a confidence level of 99.99%.

Table 3.8 SAS ANOVA Results for the Model Developed for FDBIT Pavements

Source	Degrees of Freedom	Sum of Squares	Mean Square	F Value	Prob > F
Model	2	1.29274	0.6463	320.03	0.0001
Error	37	0.07473	0.0020		
Total	39	1.36747			

Root MSE: 0.04494 R-square: 0.8127
Dep. Mean: 0.15758 Adj. R-sq: 0.8095
C.V. 28.51995

Parameter Estimates

Variable	Deg. of Freedom	Parameter Estimate	Standard Error	T for Ho: Parameter = 0	Prob > {T}
AGE	1	0.021872	0.00189	11.56	0.0001
THICKNESS	1	0.001025	0.00099	1.034	0.0176

PDBIT Pavements: For the PDBIT pavements, the selected model is:

$$SN = 0.0166 * age \qquad (3.7)$$

The R^2 value for this model is 0.7195 and the significance (p) value for the parameter age is 0.0001; i.e., the variable age is significant at a level more than 99%. The ANOVA results in Table 3.8 for this model indicates that the model has an F-value of 842, and its significance value is 0.0001. Since the selected model also has a high F-value and a very low p-value, it satisfactorily passes the model utility test. Also the estimated root mean square error ( ) value for the model is 0.046, which reveals that the selected model will predict the decrease in structural number at a variability of ±2 or ±0.092 with a confidence level of 99%.

The FDBIT and PDBIT models indicate that a 25-mm (1.0-inch) AC overlay with a structural

Table 3.9 SAS ANOVA Results for the Model Developed for PDBIT Pavements

<table>
<tr><th colspan="2">Source</th><th>Degrees of Freedom</th><th>Sum of Squares</th><th>Mean Square</th><th>F Value</th><th>Prob > F</th></tr>
<tr><td colspan="2">Model</td><td>1</td><td>1.84718</td><td>1.84718</td><td rowspan="3">841.8</td><td rowspan="3">0.0001</td></tr>
<tr><td colspan="2">Error</td><td>84</td><td>0.18432</td><td>0.00219</td></tr>
<tr><td colspan="2">Total</td><td>85</td><td>2.03150</td><td></td></tr>
<tr><td colspan="7">Root MSE: 0.04684 R-square: 0.7195
Dep. Mean: 0.14286 Adj. R-sq: 0.7098
C.V.: 32.79012</td></tr>
<tr><th colspan="7">Parameter Estimates</th></tr>
<tr><th>Variable</th><th>Deg. of Freedom</th><th>Parameter Estimate</th><th>Standard Error</th><th colspan="2">T for Ho: Parameter = 0</th><th>Prob > {T}</th></tr>
<tr><td>AGE</td><td>1</td><td>0.016685</td><td>0.000575</td><td colspan="2">29.014</td><td>0.000</td></tr>
</table>

layer coefficient of 0.42 on 200-mm (8.0-in) thick asphalt pavements will have no affect on the *decrease* of the structural number of the pavement in about 19 and 25 years, respectively, for these two types of pavement. In other words, the fatigue lives of these AC layers will be fully consumed by that time. According to the algorithms in HYNELIFE, in 10 years the *decrease* in structural number of this *overlay* would be 0.08 *(= 0.42-0.34)*. Moreover, the *decrease* in the structural number of a 25-mm (1-inch) AC layer which has been overlaid two times over a period of 20 years *(one overlay every 10 years)* is 0.28 (i.e., SN=0.28). However, the models in this study (Equations 3.6 & 3.7) show that after 20 years, on average, the decrease in structural number of a 25-mm (1-inch) overlay would be 0.42. Thus, these models overestimate the damage by 0.42/0.28 (= *150%)* or 50% *higher* compared to the assumptions in HYNELIFE.