AUBURN

SAMUEL GINN
COLLEGE OF ENGINEERING

Final Report

EVALUATION OF FLEXIBLE PAVEMENT PERFORMANCE USING LTPP DATA

Submitted to

Alabama Department of Transportation
Research Project 930-419

Prepared by

Frazier Parker
Jiansheng Song

July 2002

Highway Research Center
Harbert Engineering Center
Auburn, Alabama 36849

www.eng.auburn.edu/research/centers/hrc.html

EVALUATION OF FLEXIBLE PAVEMENT PERFORMANCE USING LTPP DATA

FINAL REPORT
PROJECT 930-419

By
Frazier Parker
Jiansheng Song

Highway Research Center
Auburn University, Alabama

Sponsored by
Alabama Department of Transportation
Montgomery, Alabama

July 2002

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

Pavements are complicated physical structures responding in a complex way to the influence of many variables. Understanding how the long-term performance of pavement relates to the factors such as environmental and traffic loads is key to building and maintaining a cost-effective highway system. For this purpose, SHRP-LTPP, a comprehensive 20-year study, was designed to evaluate the performance of in-service pavements.

In this research, pavement performance related data available in DataPave3.0 were extracted and synthesized to carry out a systematic study of the variables affecting performance of pavements.

Firstly, GPS-1 and GPS-2 test sections from Alabama were analyzed to evaluate the performance predictive capability of the 1993 AASHTO flexible pavement design model. Research showed that, statistically speaking, the 1993 AASHTO flexible pavement design model provides biased prediction of the performance of flexible pavements with granular bases and unbiased prediction of the performance of pavements with bound bases when using subgrade resilient moduli from laboratory testing. Research revealed serviceability degradation and subgrade resilient modulus are the key parameters for the 1993 AASHTO design model.

Secondly, 48 SPS-1 sections in Alabama, Arizona, Arkansas, and Iowa were used to evaluate the effects of specific design feature on pavement performance. Research revealed that fatigue cracking was a serious distress manifestation for

pavements with granular bases. "Full-depth" AC pavements performed better than other structural designs. Open graded drainable base layers improved the performance of pavements over fine-grained subgrades but pavements with dense graded black bases over coarse-grained subgrades performed better.

Thirdly, 40 SPS-5 sections in Alabama, Alberta, California, Mississippi and Texas were chosen to evaluate strategies for rehabilitation of existing pavements. Research indicated that milling of existing pavement surfaces before overlay provided some benefits for thin overlay but no benefits for thick overlay. Virgin AC mixes performed consistently better than AC mixes with 30% RAP. Increased overlay thickness (thus total thickness) improved performance, especially in preventing fatigue and transverse cracking.

Data in the SHRP-LTPP database provided a good platform for conducting this research. But, it has deficiencies such as incompleteness of certain data, and inconsistencies of some data elements which certainly diminishes confidence in the evaluations performed.

CHAPTER 1:

INTRODUCTION

1.1 PROBLEM STATEMENT

Pavements are complicated physical structures responding in a complex way to the influence of many variables such as loading, environment, material properties and variability, construction quality, maintenance etc. Evaluations of these influences are of great interests to pavement engineers. Pavement engineers consistently strive for better design procedures and strategies for the rehabilitation of existing pavements. To satisfy these needs, the Strategic Highway Research Program – Long Term Pavement Performance (SHRP-LTPP), a comprehensive 20-year study, was designed to evaluate the performance of in-service pavements.

DataPave3.0, which provides a user-friendly format for exploring, extracting, and organizing SHRP-LTPP information, including information on actual pavement performance through 2001, permits preliminary evaluation of the variables affecting performance of pavements, design procedures, and rehabilitation strategies. The pavement design engineer can use the data to check the validity of the design procedures and the appropriateness of the various assumptions that are made during the design process. The material engineers can verify whether a given type of material is appropriate for the expected level of load and anticipated environmental conditions. Pavement management engineers can make sensible recommendations for various maintenance alternatives for specific applications, which is becoming an increasingly critical task since highway agencies at all levels (city, county, and state) are generally operating under a limited budget that requires effective prioritization and use to provide the highest level of public service.

1.2 OBJECTIVES OF STUDY

This study was conducted to: 1) evaluate the 1993 AASHTO flexible pavement design model, 2) determine the effects of specific design feature on pavement performance, and 3) evaluate several strategies for rehabilitation of existing flexible pavements, using the data from the SHRP-LTPP program.

1.3 SCOPE OF STUDY

As ninety-six percent of all paved roads and streets in the U.S. (similar situation all over the world) - almost two million miles - are surfaced with asphalt, this research focused on flexible pavements.

Firstly, pavements from GPS-1 (Asphalt Concrete on Granular Base) and GPS-2 (Asphalt Concrete on Bound Base) studies in Alabama were investigated. Material properties, traffic data, and performance data were synthesized from DataPave3.0 to evaluate the predictive capability of the 1993 AASHTO flexible pavement design model, by comparing predicted pavement performance with actual (measured) performance..

Secondly, based on the availability of meaningful and sufficient performance data, SPS-1 (Strategic Study of Structural Factors for Flexible Pavements) sites in

Alabama, Arizona, Arkansas, Iowa were chosen to study the influences of the following design features:

1. The influence of surface course thickness (other factors being similar) on pavement performance;
2. The influence of drainable base layers (other factors being similar) on pavement performance;
3. Difference in performance of pavements with bound and unbound base layers (other factors being similar).

Lastly, based the availability of meaningful and sufficient performance data, SPS-5 (Rehabilitation of Asphalt Concrete Pavements) sites in Alabama, Alberta, California, Mississippi, and Texas were chosen to study the following design and construction features for hot mix asphalt overlays of flexible pavements:

1. Performance of different overlay thicknesses;
2. Performance of overlays on milled and non-milled surfaces, to determine the usefulness of milling in overlay construction; and
3. Performance of overlays constructed with mix that contains RAP and mix without RAP.

Comparisons of the performance of pavements were based on roughness (International Roughness Index or IRI), rutting, fatigue cracking (FC), longitudinal cracking (LC) and transverse cracking (TC) increases with traffic/time.

CHAPTER 2:

EVALUATION OF AASHTO FLEXIBLE PAVEMENT DESIGN EQUATION

The 1993 AASHTO flexible pavement design model is semi-empirical. It is satisfactory as long as the materials, traffic loading conditions, and layer thickness do not significantly differ from those for which the model was developed. Because of considerable increase in frequency and intensity of axle loads and the use of new pavement construction materials, the design model needs to be evaluated and possibly modified to improve its predictability. This research used data from Alabama GPS-1 and GPS-2 sections for this evaluation. Preliminary analysis included Alabama SPS-1 sections in the evaluation, but they were deleted because pavement condition degradation was minimal and DataPave3.0 contained no traffic data.

2.1 EVALUATION PROCEDURE

The 1993 AASHTO design equation (Part I: section 1.2) for flexible pavements is as follows:

$$\begin{aligned} \mathrm{Log}_{10}(W_t) = {} & Z_R{*}S_0 + 9.36{*}\log_{10}(SN+1) - 0.2 \\ & + \log_{10}(\Delta PSI/(4.2-2.7)) \,/\, (0.4 + 1094 \,/\, (SN+1)^{5.19}) \\ & + 2.32{*}\log_{10}(M_R) - 8.07 \qquad \text{------ Eq. 2.1} \end{aligned}$$

Where:

W_t = predicted number of 18-kip equivalent single axle load application,

Z_R = standard normal deviate, which is a function of the reliability level for the design,

S_0 = overall standard deviation for the traffic and performance prediction,

ΔPSI = difference between the initial serviceability index, p_i, and the terminal serviceability index, p_t, and

M_R = subgrade resilient modulus (psi).

The following procedure was applied to accomplish this task.

1) Available pertinent data were obtained from the LTPP database, i.e., DataPave3.0;
2) The data were interpreted to determine required inputs for the 1993 AASHTO flexible pavement design model (e.g., layer thicknesses, initial serviceability, measured serviceability, changes in serviceability, resilient modulus for the subgrade and structural number). The initial serviceability was estimated by extrapolation using historical IRI data. The subgrade resilient modulus was obtained by averaging the laboratory results for several test conditions. For comparison purpose, subgrade resilient modulus was also estimated by backcalculating modulus from Falling Weight Deflectometers (FWD) data with EVERCAL5.0 and DARwin procedures. The structural number was obtained using typical layer coefficients for various type materials and layer thicknesses;
3) The predicted cumulative ESALs (W_t) were calculated with the AASHTO equation;

4) Predicted performance was compared with observed performance by comparing PSI-Wt curves; and
5) Statistical comparisons were made between predicted and observed cumulative ESALs for the final observed ΔPSI to evaluate the AASHTO equation. Observations were made, e.g., on selection of the design inputs under certain circumstances to improve the model's predictive capacity. This process was applied on all of the sections and sections with different base types.

2.2 DATA PREPARATION

As most of the inputs required for the 1993 AASHTO flexible pavement design equation need to be estimated or interpreted from other data elements available in DataPave3.0, a detailed discussion of the inputs is included below. The discussion specifically addresses the availability of the data elements, the assumptions and procedures used, and the limitations of each data set.

2.2.1 Structural Number (SN)

The structural number was calculated for each LTPP section using the following equation presented in the 1993 AASHTO guide (Part I: Section 1.2):

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \text{ .... Eq. 2.2}$$

where:

SN = structural number indicative of the total pavement thickness,

a_i = ith layer coefficient,

D_i = ith layer thickness (in), and

m_i = ith layer drainage coefficient.

Because moduli from laboratory tests and/or FWD backcalculation for asphalt concrete and bases/subbase layers are not considered sufficiently reliable to differentiate between specific materials, the following layer coefficients were assigned based only on material type:

AC – Asphalt concrete surface and binder, $a_1 = 0.44$;

DGBB – Dense graded black base, $a_2 = 0.34$;

OGBB – Open graded black base, $a_2 = 0.20$;

GRB – Granular base, $a_2 = 0.15$;

GRSB – Granular subbase, $a_3 = 0.11$.

Two different sources of layer thickness are available in the LTPP database; 1) design thickness from inventory data, and 2) mean core thickness from testing data. In this analysis, the mean core thicknesses were considered first, and inventory data were used as a supplement when testing data were not available, or when they appeared unreasonable.

Because data were not available to definitively characterize drainage conditions of base/subbase layers, all layer drainage coefficients were assigned as 1.0.

2.2.2 Serviceability

As initial serviceability values are not included in the LTPP database, they were estimated by backcasting the initial IRI using the historical IRI versus time data. IRI was converted to serviceability using the following equation.

$$PSI = 5 * EXP[-0.29254 * IRI] \quad \text{.......... Eq. 2.3}$$

Where:

PSI – Present service index, and

IRI – International roughness index (in units of m/km).

2.2.3 Subgrade Resilient Modulus

Subgrade resilient modulus can be obtained from 1) laboratory testing results, 2) backcalculation using EVERCAL5.0, and 3) backcalculation using DARwin. As subgrade resilient modulus is an important parameter for the 1993 AASHTO design model, detailed comparisons are needed to determine which one to be used. This will be discussed later.

2.2.4 Reliability Level (%) and Overall Standard Deviation

A reliability level of 50% was used in evaluations. This is in essence a neutral assumption that elimates bias in predictions. The standard normal deviate, Z_R, is zero and, therefore the term Z_R*S_0 is also zero. This eliminates consideration of reliability from the equation.

2.2.5 Actual LTPP Traffic Data

Annual historical ESAL data were obtained from the database and plotted versus time for all available sections. Linear regression equations through ESAL data were used to estimate the traffic volume of those years with no data. The total accumulated ESALs, from the traffic opening date to dates at which IRI were measured, were obtained by summing annual ESALs computed with the regression equation for each year. The traffic data were highly variable and very limited for some sections and are potentially a major source of difference between predicted and observed performance.

2.3 ANALYSIS OF RESULTS

2.3.1 Selection of Input Parameters for AASHTO Design Equation

An example of parameters for GPS-2 sections is shown in Table 2.1. Tabulated values include the original construction date and when applicable, overlay dates. These data were used to extrapolate (backwards) regression equations to estimate

Table 2.1 Parameters for Alabama GPS-2 Section 1021

Construction Date				6/1/85
SN	Layers	D_i(in)	a_i	4.81
	AC	3.1	0.44	
	DGBB	4.5	0.34	
	GRSB	17.4	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	6/1/85	0.89	3.9	0
	6/6/90	0.96	3.77	505000
	4/3/92	0.95	3.79	757000
	2/17/94	0.99	3.74	1047000
	12/13/95	1.01	3.73	1340000
M_R	Evercalc			9280 psi
	DARwin			7250 psi
	Lab			10585 psi

initial PSI values and total accumulated ESALs. Layer thicknesses were measured from coring and drilling. Layer coefficients are "typical values" as previously described, and the structural number was computed with these "typical values" for layer coefficients and measured layer thicknesses.

Initial (at zero accumulated ESALs) estimated IRI and, thus PSI are highlighted in Table 2.1. Dates where IRI were measured and where corresponding PSI were computed are shown with estimated accumulated ESALs. Estimated subgrade moduli backcalculated from FWD data with Evercalc5.0 and DARwin are shown with laboratory measured values.

Figure 2.1 illustrates the procedure to estimate the initial IRI with linear regression from measured IRI data points for GPS-2 section 1021. Equation 2.3 was used to estimate initial PSI from initial IRI. There were several sections where linear regressions did not fit well all measured IRI. In these cases, linear regression through a subset of points was used to extrapolate backwards to an initial IRI.

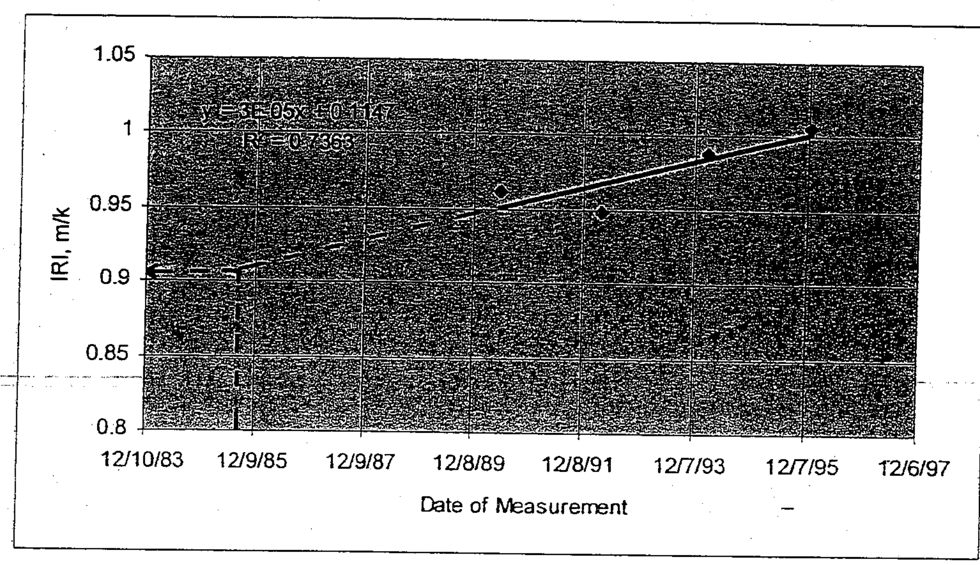


Figure 2.1 Regression Equation for Backward Extrapolation to Estimate Initial IRI for Alabama GPS-2 Section 1021

Figure 2.2 shows a regression equation through estimated annual ESALs for GPS-2 section 1021. For traffic, linear regressions were used primarily to extrapolate forward in time, rather than backwards, to estimate the total accumulated traffic by summing annual ESALs for years in service.

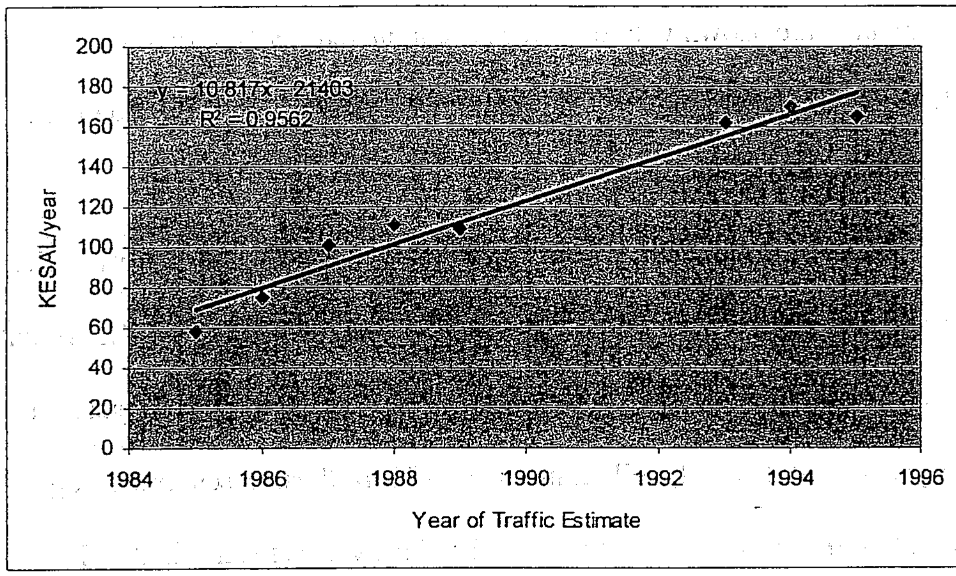


Figure 2.2 Regression Equation for Estimating Accumulated ESAL's for Alabama GPS-2 Section 1021

Figure 2.3 shows several relationships between PSI and W_t (accumulated ESALs) for GPS-2 section 1021. One is the measured relationship, i.e., PSI computed from measured IRI with estimated accumulated ESALs. The initial PSI computed with the estimated initial IRI is also shown as part of the relationship. The other three

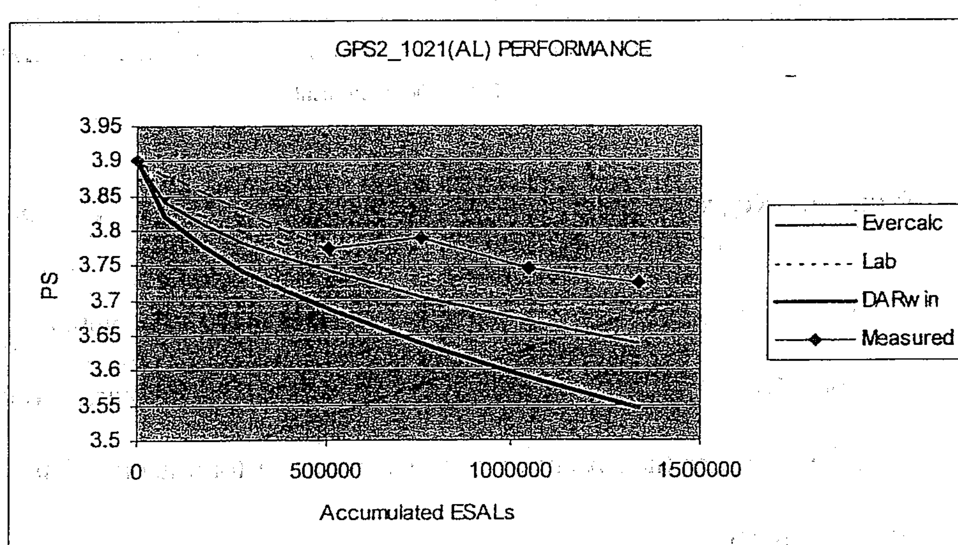


Figure 2.3 PSI-W_t Curve for Alabama GPS-2 Section 1021

relationships were computed with Equation 3.1 using the three estimates of subgrade resilient moduli shown in Table 2.1. Tables, similar to Table 2.1, and figures, similar to Figure 2.3, were prepared for all Alabama GPS-1 and GPS-2 sections and are shown in the **APPENDIX**.

Figure 2.3 indicates that subgrade resilient modulus may have considerable influence on accumulated ESALs predicted with the AASHTO design equation. An examination of figures in the **APPENDIX** for other GPS-1 and GPS-2 sections confirm this observation, and that the use of laboratory subgrade resilient moduli produces curves that are, in most cases, more comparable with measured relationships. These comparisons and studies of backcalculated and laboratory subgrade resilient moduli for Alabama SPS-1 and SPS-5 sections lead to the use of laboratory values for evaluating the AASHTO design equation.

Studies for SPS-1 and SPS-5 sections indicate that backcalculated subgrade resilient moduli are influenced by the stiffness of the pavement structure (SN). SPS sections are constructed on roadway sections of limited length where relatively uniform subgrade support would be expected. In the case of SPS-1 sections, positive steps were taken during construction to insure uniform subgrade support. Laboratory moduli in Figure 3.4 show, as expected, relatively uniform support for SPS-1 sections ($M_R \approx 10{,}000$ psi). Moduli backcalculated with DARwin, however, are strongly influenced by and correlated with ($R^2 = 0.7118$) SN. Laboratory moduli are not available for SPS-5 sectios, but the strong influence of SN on backcalculated subgrade resilient moduli is illustrated in Figure 2.5.

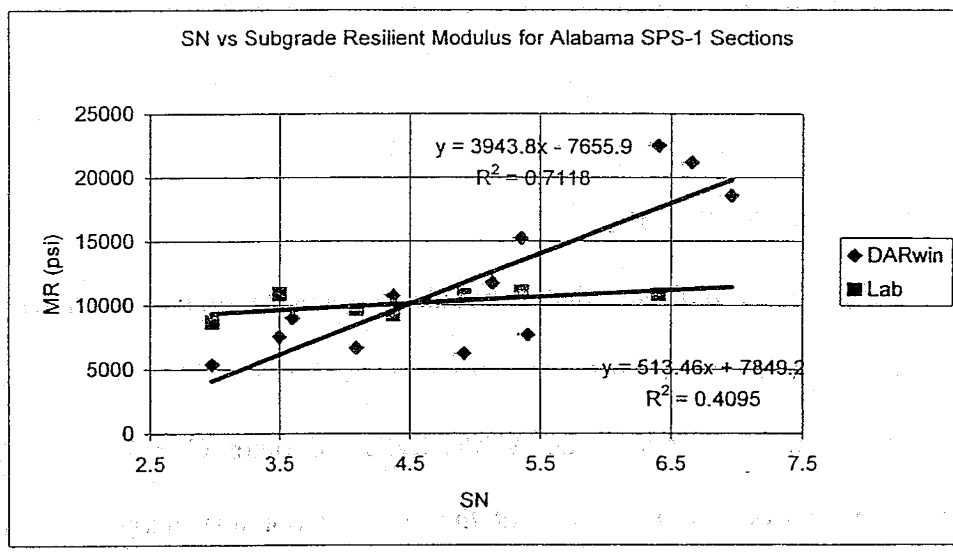


Figure 2.4 Relationship between SN and Subgrade Resilient Modulus for Alabama SPS-1 Sections

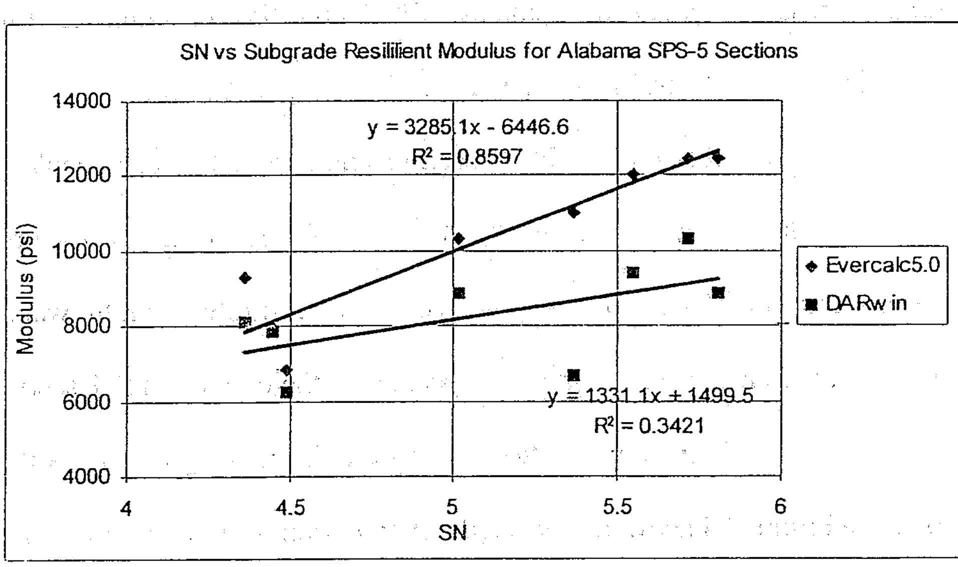


Figure 2.5 Relationship between SN and Subgrade Resilient Modulus for Alabama SPS-5 Sections

The observed relationships between SN and backcalculated M_R are likely the result of stress sensitivity of the subgrade soils, i.e., higher SN results in lower subgrade stresses which would give higher stiffness. This would mean that backcalculated M_R are more representative of existing subgrade support and, therefore, should provide better predictions of performance with AASHTO equation. This was not the case for GPS-1 and GPS-2 sections, as previously noted, and was not the case for SPS-1 sections. Figure 2.6 is an example of measured and computed PSI-accumulated ESALs relationships for SPS-1 sections. Figure 2.6 is typical of 10 of 12 sections and confirms the validity of using laboratory subgrade resilient moduli for evaluating the AASHTO equation. Although the performance comparisons are limited and more extensive evaluations are certainly needed, implications seem to be that FWD loadings do not stress subgrade soils to levels experienced under traffic. Table 2.2 summarizes SN and subgrade M_R values for GPS-1 and GPS-2 sections.

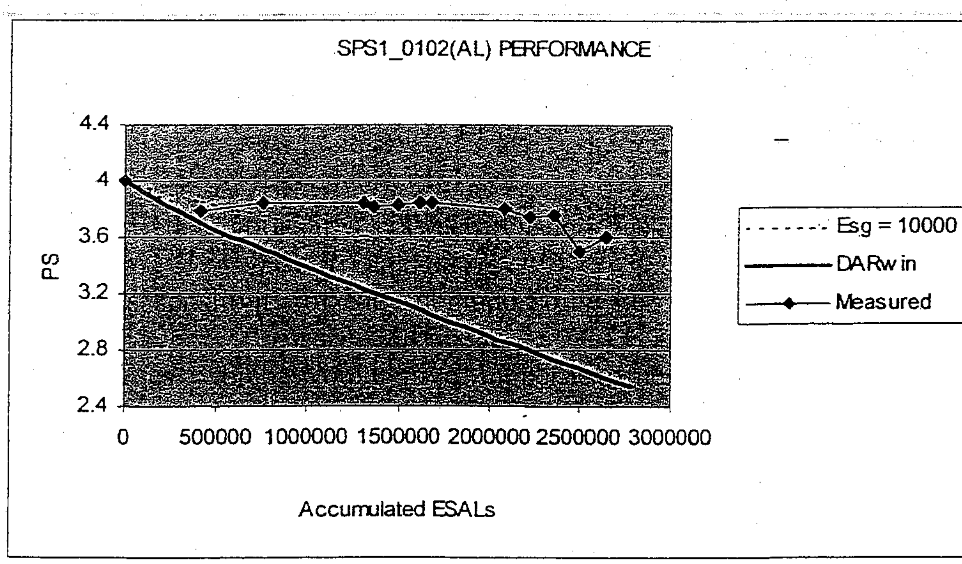


Figure 2.6 PSI-W_t Curve for Alabama SPS-1 Section 0102

Table 2.2 Structural Numbers and Subgrade Resilient Moduli for Alabama GPS-1 and GPS-2 Sections

Sections		SN	Lab M_R (psi)
GPS-1	1001	4.48	26,390
	4126	7.79	9,715
	4127	5.51	13,340
	4129	4.53	11,165
	4155	3.19	17,690
GPS-2	1011	3.95	12,899
	1019	3.17	12,180
	1021	4.81	10,850
	4073	4.14	11,890
	4125	4.32	13,050

2.3.2 Evaluation of the Performance Predictive Capability of the 1993 AASHTO Flexible Pavement Design Model

The 1993 AASHTO flexible pavement design model was evaluated by comparing predicted ESALs with actual ESALs carried to produce an observed loss of PSI. The predicted ESALs from the AASHTO model were compared with the estimated ESALs carried by the sections when the last IRI (and thus PSI) were measured. Plots of predicted versus actual ESALs and regression equations were constructed. Then, paired t-tests were carried out to determine statistical significance of differences. The null hypothesis is that the difference between mean predicted $\log W_t$ and mean actual $\log W_t$ is equal to zero, and the alternate hypothesis is that it is not equal to zero (the model over or under predicts $\log W_t$). This was to evaluate the predictions for bias. Finally, the standard deviation of the differences between predicted $\log W_t$ and actual $\log W_t$ was computed for each plot to compare with the value (0.49) contained in the 1993 AASHTO design guide.

Table 2.3 shows results computed with the 1993 AASHTO design model. Column 2 contains the relationship between $\log W_t$ and ΔPSI for each section. Structural numbers and subgrade resilient moduli used to develop the equations are tabulated in Table 2.2. Column 3 contains the estimated initial PSI and the PSI computed with last measured IRI for each section. Column 4 shows the PSI changes ($\Delta PSI = PSI_i - PSI_f$). Column 5 contains predicted $\log W_t$ when ΔPSI in column 4 is substituted into the equations in column 2. Column 6 contains the log of accumulated ESALs at the time the last IRI was measured.

Table 2.3 Results of Applying the 1993 AASHTO Flexible Pavement Design Model for GPS-1 and GPS-2 Sections in Alabama

Sections	AASHTO Equation	PSI_i/ PSI_f	ΔPSI	Predicted $\log W_t$	Actual $\log W_t$
		GPS-1			
1001	$\log W_t = 1.64 \log \Delta PSI + 7.98$	4.20/3.83	0.37	7.27	6.37
4126	$\log W_t = 2.41 \log \Delta PSI + 8.68$	4.00/3.71	0.29	7.38	7.03
4127	$\log W_t = 2.15 \log \Delta PSI + 7.99$	3.90/3.71	0.19	6.44	6.07
4129	$\log W_t = 1.81 \log \Delta PSI + 7.29$	3.70/2.82	0.88	7.19	6.13
4155	$\log W_t = 0.96 \log \Delta PSI + 6.99$	4.00/3.63	0.37	6.58	6.35
		GPS-2			
1011	$\log W_t = 0.99 \log \Delta PSI + 6.71$	4.00/3.78	0.22	6.06	5.97
1019	$\log W_t = 0.94 \log \Delta PSI + 6.60$	3.50/2.96	0.54	6.35	6.28
1021	$\log W_t = 1.93 \log \Delta PSI + 7.41$	3.90/3.73	0.17	5.92	6.13
4073	$\log W_t = 1.60 \log \Delta PSI + 7.15$	4.00/3.74	0.26	6.21	6.24
4125	$\log W_t = 1.70 \log \Delta PSI + 7.33$	4.30/3.42	0.88	7.24	6.81

2.3.2.1 All Pavements

A plot of predicted versus actual log(ESALs) for all Alabama test sections (data in Table 2.3) is presented in Figure 2.7. Figure 2.7 indicates the AASHTO equation with laboratory subgrade resilient modulus over-predicts the pavement performance of Alabama GPS-1 and GPS-2 pavements.

Table 2.4 contains results of a t test comparing mean actual $\log W_t$ with mean predicted $\log W_t$. The computed t is greater than the critical t at the 0.05 level of significance, which means the null hypothesis can be rejected, i.e., generally speaking, the 1993 AASHTO flexible pavement design model provides biased prediction of flexible pavement performance. The design model over-predicts performance by 109%.

The standard deviation for the differences between predicted and actual $\log W_t$ values was 0.40, as compared to the value 0.49 for flexible pavements from the AASHTO Road Test. This indicates that the model is less variable in prediction than expected. This variability may be due to:

1) Traffic estimation: This is potentially the greatest source of error because the process of obtaining "actual" ESALs depends on many variables that are difficult to estimate. Sources of error include estimating annual traffic volumes for years without data with regression equations, volumes of each axle type, lane distribution of trucks, and directional distributions of trucks.
2) Design input: Errors may be introduced when backcasting the initial IRI and the estimation of PSI values using the PSI-IRI relationship. Another important source of error comes from the subgrade resilient modulus, which is also often inaccurate

and fails to represent the real subgrade conditions. Structural numbers for pavement structures were computed with AASHTO recommended layer coefficients and this may introduce errors if material properties were much different from the norm at the AASHTO road tests.

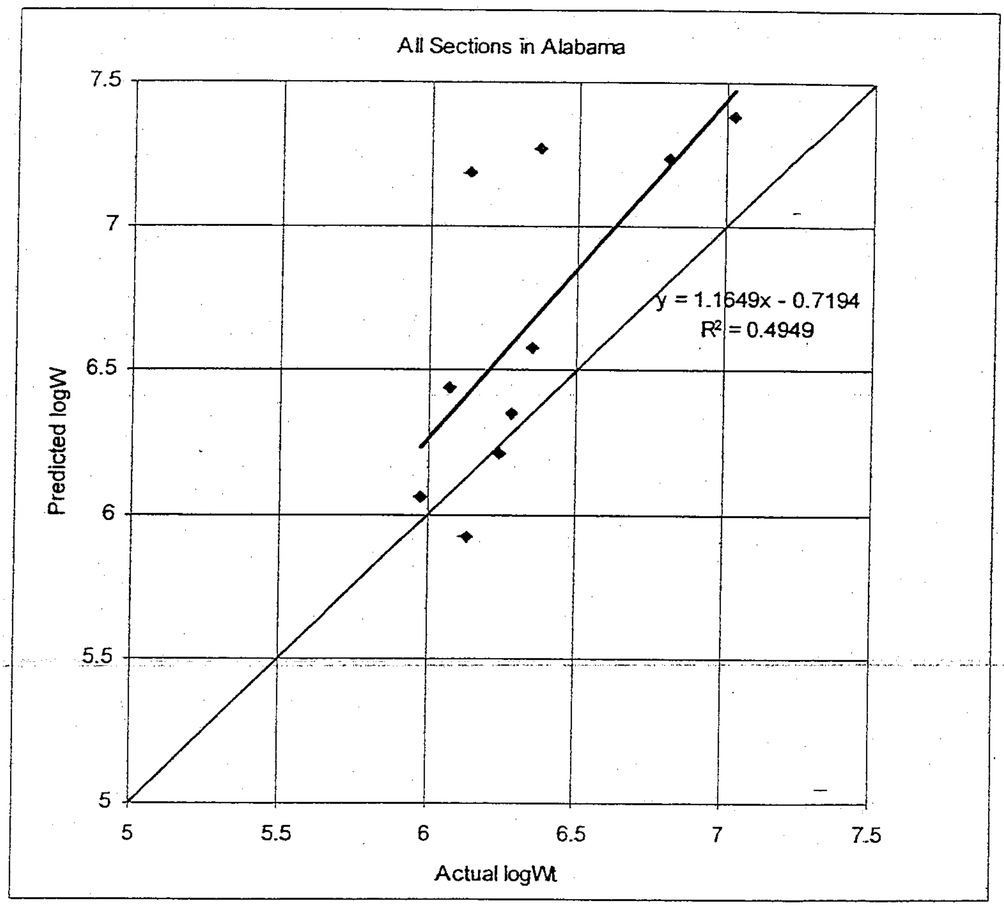


Figure 2.7 Relationship between Predicted logWt and Actual logWt for All Test Sections in Alabama

Table 2.4 Statistics Analysis for All Test Sections in Alabama

Category	No. of Sections	Mean Actual $\log W_t$	Mean Pred. $\log W_t$	t - value	t-critical (0.05 level)	Rejected?	SD*
All	10	6.34	6.66	2.56	2.26	YES	0.40

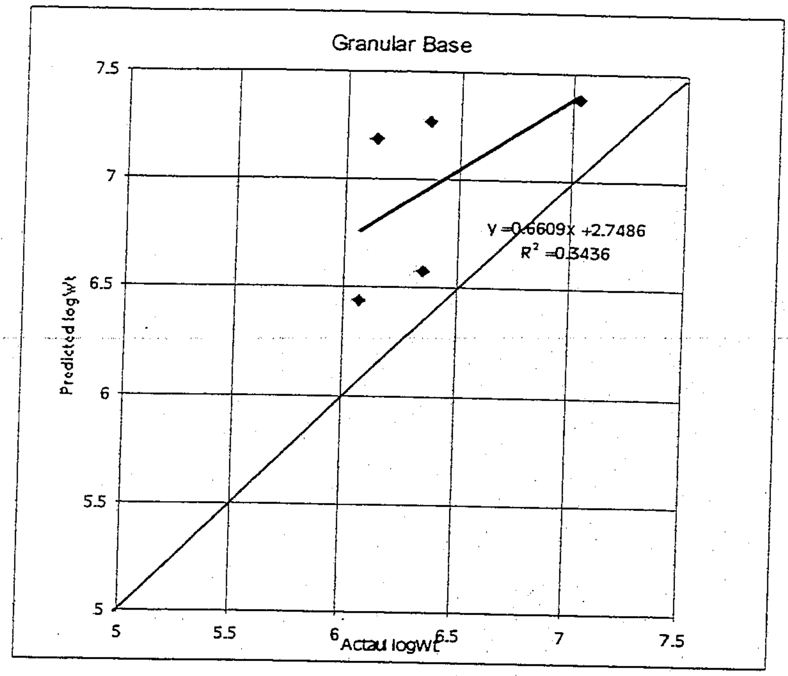
* Standard deviation of the difference between the actual and predicted $\log W_t$

2.3.2.2 Pavements with Granular Bases versus Those with Bound Bases

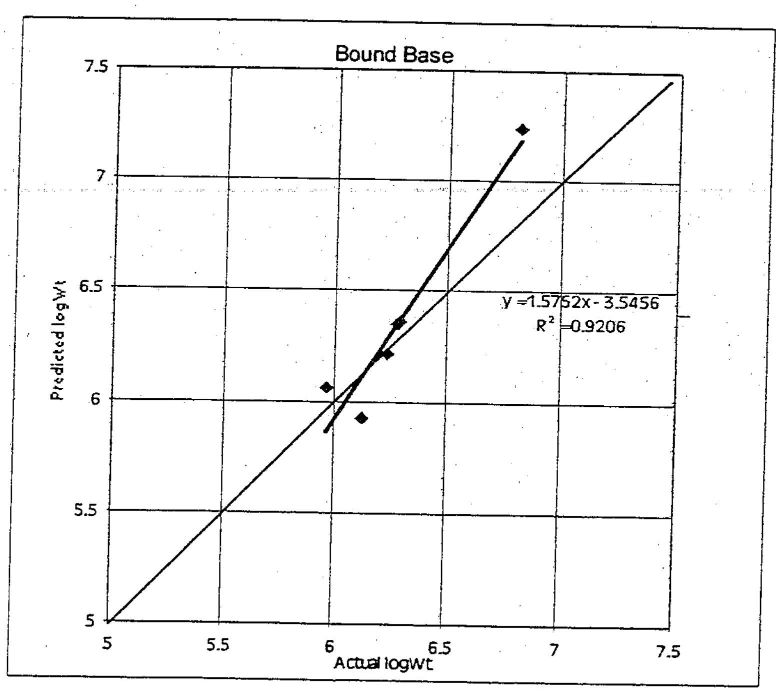
Plots of predicted versus actual log(ESALs) for pavements with granular bases are presented in Figure 2.8 (a) and for pavements with bound bases in Figure 2.8(b). These figures indicate the design model over-predicts performance for both groups but predictions are better for pavements with bound bases.

The paired t-test was used to evaluate the results for bias and the complete results are shown in Table 2.5. Table 2.5 shows that, for pavements with granular bases, the computed t is greater than the critical t at the 0.05 level of significance, which means the null hypothesis can be rejected, i.e., generally speaking, the 1993 AASHTO flexible pavement design model provides biased over prediction of performance. On the contrary, for pavements with bound bases, the prediction is unbiased.

The performance of pavements with granular bases is over-predicted by 280% while the performance of pavements with bound bases is over-predicted by only 17%. Standard deviations for differences between predicted $\log W_t$ and actual $\log W_t$ values were 0.37 and 0.23, which are less than the value of 0.49 for AASHTO test data used to develop the model. The 1993 AASHTO flexible pavement design model was developed from data for pavements that had experienced large serviceability loses as they approached terminal levels. However, the pavements analyzed had experienced only relatively small serviceability loses and may be one reason for the large biased predictions.



(a)



(b)

Figure 2.8 Relationship between Predicted logWt and Actual logWt for Pavements with Different Base Types

Table 2.5 Statistics Analysis for Pavements with Different Base Types

Base Type	No. of Sections	Mean Actual $\log W_t$	Mean Pred. $\log W_t$	t-value	t-critical (0.05 level)	Rejected?	SD*
Granular	5	6.39	6.97	3.50	2.78	YES	0.37
Bound	5	6.29	6.36	0.67	2.78	NO	0.23

* Standard deviation of the difference between the actual and predicted $\log W_t$

CHAPTER 3:

PERFORMANCE EVALUATION OF SPS-1 PAVEMENTS

Generally speaking, pavements fail structurally because of a combination of three primary factors: traffic (cumulative wheel load application), inadequate structural capacity (or loss of structural capacity), and poor support (or loss of support). SPS-1 sections at each site were constructed on similar subgrades within the same climatic region and carry the same traffic. As a result, they provide a good opportunity to study the influence of pavement structural parameters on performance. The study focused on SPS-1 sites in Alabama, Arizona, Arkansas, and Iowa. Table 3.1 summarizes basic information for sites in these states.

Table 3.1 Basic Information about SPS-1 Sites

State	Age (year)	Accumulated Traffic (KESAL)	Climatic Region	Subgrade Type
Alabama	9	4230	W-NF	Fine
Iowa	7	N/A*	W-F	Fine
Arizona	7	N/A*	D-NF	Coarse
Arkansas	6	1500	W-NF	Coarse

*N/A – not available

SPS sections were designed to permit direct performance comparisons among sections. For this research, sections were regrouped according to their specific design features. All performance comparisons were based on smoothness degradation, rutting, fatigue cracking, longitudinal cracking and transverse cracking increases with traffic/time. For convenience of comparison, a distress index (DI) was determined by summing all distress measurements. DI is defined as follows:

$$DI = 100 * \Delta IRI + Rutting + FC + LC + TC \qquad \text{------ Eq. 3.1}$$

Where:

DI = distress index,

ΔIRI = IRI change (m/km),

Rutting = rut depth (mm),

FC = fatigue cracking (m^2/section),

LC = longitudinal cracking (m/section), and

TC = transverse cracking (no./section).

It should be noted that DI is an artificially defined index with no specific meaning (or no units). ΔIRI is amplified by 100 times because of its importance as an indicator of performance and so that its magnitude would be comparable to the magnitude of rutting and cracking measurements. The performance of test sections was considered different if their DIs differed more than 50.

The following procedures were applied to accomplish comparisons of the performance of test sections:

1) Detailed data about layer type, layer thicknesses of each section were synthesized;
2) Sections were regrouped for comparison purposes based on their surface layer thicknesses, base types, and base thicknesses;

3) Comparisons based on performance were performed within each group for sections with similar structural number (SN);

4) Comparisons based on performance were performed among different sites; and

5) Relationships and conclusions based on the comparisons were developed.

For consistency, symbols for the several type pavement structures are defined as follow:

Layers	Symbol	Layers	Symbol
Surface Granular base Subgrade	**G**	Surface Dense Graded Black Base Granular Subbase	**GB**
Surface Open Graded Black Base Granular Subbase Subgrade	**GO**	Surface Dense Graded Black Base Open Graded Black Base Geotextile	**TOB**

3.1 ANALYSIS OF RESULTS

3.1.1 Comparisons within Groups at Each Site

Table 3.2, 3.3, 3.4 and 3.5 contain pavement descriptions and performance parameters for the SPS-1 sections in Alabama, Iowa, Arizona, and Arkansas respectively, showing the groups and the corresponding performance data. Direct comparisons of the performance of sections are also included in the tables. Comparisons are made to determine the effects of surface course thickness, an open graded drainable layer and base type (granular, open graded or dense graded black base). These comparisons are made between sections with similar structural numbers and the validity of comparisons will certainly depend on structural number similarity. Conclusions from the comparisons can be summarized as follows:

1) Asphalt concrete surface thickness

Constructed on granular base, pavements with thicker AC surface showed somewhat better performance (especially less fatigue cracking) than those with thinner surface course.

2) Drainable layer

Pavements with drainable layers performed better than those without drainable layers on fine-grained subgrades (Alabama and Iowa). However, for coarse-grained subgrades, pavements with dense graded bases performed better than pavements with drainable bases (Arizona and Arkansas).

3) Base type

Dense graded black base layers improved pavement performance when used alone (full depth) or in combination with open graded or granular base layers.

3.1.2 General Observations

Comparisons between states may not be meaningful because of differences in accumulated traffic and the fact that accumulated traffic in Iowa and Arizona are unknown. However, the following general observations are offered:

1) Fatigue cracking is more prevalent in Alabama pavements (52 m^2/section) than the other three states (15, 17.6 and 18 m^2/section). And, it is more prevalent in pavements with granular bases.
2) Rutting depths are larger in Arizona (17.6 mm) than in the other three states (10.4, 6.6 and 7 mm) and may be due to higher temperature.

3) Transverse cracking is more prevalent in Iowa (average of 18.4 no./section) than the other three states (average of 1.3 no./section) and confirms that transverse cracking is cold temperature related distress.

4) Longitudinal cracking is most prevalent in Arizona (62 m/section) and least prevalent in Arkansas (4 m/section). The large longitudinal cracking in Arizona may be related to the large rutting (either in or between wheel paths).

Table 3.2 Performance Comparisons among Alabama SPS-1 Sections

Section	Base	ST (in)	SN	6IRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
Surface Course Thickness									
0101	G	6.6	4.1	0.1	8	39	16	1	74
0102	G	3.9	3.5	0.38	15	145	46	1	245
Thicker surface better									
Drainage Layer									
0105	GB	4.1	3.8	0.13	10	137	37	0	197
0107	GO	4.1	3.1	0.15	15	44	20	0	94
Drainable better									
0106	GB	7.4	6.6	0.17	11	0	43	0	71
0109	GO	7.4	5.8	0.1	9	52	25	0	96
No apparent difference									
0102	G	3.9	3.5	0.38	15	145	46	1	245
0107	GO	4.1	3.1	0.15	15	44	20	0	94
Drainable better									
Base Type									
0102	G	3.9	3.5	0.38	15	145	46	1	245
0105	GB	4.1	3.8	0.13	10	137	37	0	197
No apparent difference									
0103	B	4.3	4.4	0.13	9	35	31	0	88
0105	GB	4.1	3.8	0.13	10	137	37	0	197
Black base better									
0104	B	6.7	7.0	0.1	9	0	0	0	19
0106	GB	7.4	6.6	0.17	11	0	43	0	71
Black base better									
0108	GO	7.3	5.2	0.15	9	110	22	0	156
0110	TOB	6.7	5.1	0.1	10	50	28	0	98
Black base better									
Avg.	-	-	-	0.17	10.4	52	27	0	107

Table 3.3 Performance Comparisons among Iowa SPS-1 Sections

Section	Base	ST (in)	SN	6IRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
Surface Course Thickness									
0101	G	8	4.7	0.5	9	13	15	20	107
0102	G	5.1	4.0	1.4	13	82	142	46	423
Thicker surface better									
0110	OB	7.9	5.4	0.5	6	3	37	14	110
0111	OB	4.4	5.4	0.3	5	1	9	13	58
Thin surface better									
Drainage Layer									
0105	GB	3.5	3.7	0.4	8	4	132	14	198
0107	GO	3.4	2.9	0	0	0	78	0	78
Drainable better									
0106	GB	6.8	6.7	0.3	4	28	39	33	134
0109	GO	7.5	6.1	0.1	8	0	11	6	35
Drainable better									
Base Type									
0102	G	5.1	4.0	1.4	13	82	142	46	423
0105	GB	3.5	3.7	0.4	8	4	132	14	198
Black base better									
0103	B	3.8	4.5	0.4	3	5	33	18	99
0105	GB	3.5	3.7	0.4	8	4	132	14	198
Black base better									
0104	B	7.0	7.3	0.4	4	20	58	29	151
0106	GB	6.8	6.7	0.3	4	28	39	24	134
No apparent difference									
0108	GO	5.9	4.7	0.8	14	21	8	17	140
0110	OB	7.9	5.4	0.5	6	3	37	14	110
No apparent difference									
Avg.	-	-	-	0.45	6.6	15	47	18	132

Table 3.4 Performance Comparisons among Arizona SPS-1 Sections

Section	**Base**	**ST (in)**	**SN**	**6IRI**	**Rutting (mm)**	**FC (m^2)**	**LC (m)**	**TC (No.)**	**DI**
					Drainage Layer				
0118	GB	4.0	5.0	0	17	12	18	0	47
0121	GO	4.1	4.5	0	16	22	120	0	158
					Dense graded better				
0114	G	6.8	4.8	0.3	18	6	130	2	186
0119	GO	6.3	4.3	0	38	0	90	0	128
					Drainable better				
					Base Type				
0114	G	6.8	4.8	0.3	18	6	130	2	186
0117	GB	7.6	5.4	0	18	0	10	0	28
					Black base better				
0115	B	6.6	5.8	0.1	9	0	40	0	59
0117	GB	7.6	5.4	0	18	0	10	0	28
					No apparent difference				
0116	B	4.1	5.9	0.1	15	4	0	0	29
0118	GB	4.0	5.0	0	17	12	18	0	47
					No apparent difference				
Avg.	-	-	-	0.09	17.6	4	62	1	94

Table 3.5 Performance Comparisons among Arkansas SPS-1 Sections

Section	Base	ST (in)	SN	6IRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
Surface Course Thickness									
0113	G	4.1	3.0	0.1	5	12	3	4	34
0114	G	6.9	3.8	0.1	7	6	3	1	27
No apparent difference									
Drainage Layer									
0118	GB	4.1	5.0	0	6	5	7	0	18
0121	GO	4.5	4.7	0.1	7	12	15	3	47
No apparent difference									
0114	G	6.9	3.8	0.1	7	6	3	1	27
0119	GO	6.8	4.3	0.8	7	125	1	15	228
Granular better									
Base Type									
0115	B	7.0	5.6	0.1	8	6	2	0	26
0117	GB	6.9	5.0	0	8	3	1	1	13
No apparent difference									
0116	B	4.1	5.8	0	7	2	1	0	10
0118	GB	4.1	5.0	0	6	5	7	0	18
No apparent difference									
0120	GO	4.2	3.7	0.7	7	40	2	5	124
0122	TOB	4.6	4.1	0.2	7	1	0	0	28
Black base better									
Avg.	-	-	-	0.22	7	18	4	3	53

CHAPTER 4:

PERFORMANCE EVALUATION OF SPS-5 PAVEMENTS

The major problem that pavement engineers face today is not how to design and construct new pavements, but how to evaluate, maintain, and upgrade existing pavements to meet demands of higher volumes of traffic and larger loads. The traffic volumes on the primary highway system, especially in urban areas, have seen tremendous increases over the last 20 years, leading in many instances to earlier-than-expected failures of highway pavements. The aging of the Interstate Highway System and other primary systems built during the 1950s and 1960s has resulted in the expenditure of a large portion of highway funds on pavement rehabilitation. Efforts continue to develop techniques and procedures that will result in more cost-effective and longer-lasting pavement rehabilitation.

A typical asphalt pavement rehabilitation strategy involves milling and resurfacing of the existing pavement. The overlay thickness may depend on the condition of existing pavement, anticipated traffic, available funds, and etc. The SPS-5 sections at a site were constructed over similar subgrade, have the same climatic region and carry the same traffic. Therefore, they provide a good opportunity to directly compare rehabilitation strategies. SPS-5 sections at different sites permit some assessment of the influence of traffic, climate and subgrade.

This study focused on SPS-5 sites in Alabama, Alberta (Canada), California, Mississippi and Texas. Table 4.1 is a summary of general conditions for these sites.

Table 4.1 Basic Information about SPS-5 Sites

State	Age (years)	Accumulated Traffic (KESAL)	Climatic Region	Subgrade Type
Alabama	9	630	W-NF	Fine
Alberta	10	3820	D-F	Coarse
California	7	13000	D-NF	Coarse
Mississippi	9	8800	W-NF	Fine
Texas	9	N/A	D-NF	Fine

4.1 ANALYSIS OF RESULTS

Three aspects of asphalt pavements rehabilitation strategies, i.e., milling of existing surface, overlay material type, and overlay thickness, were included in SPS-5 sections. Sections were grouped and comparisons made of their performance to evaluate the effects of milling and the influence of 30% RAP in overlay mixes. It should be noted that no data was found to determine if mix designs for mixes with and without RAP were comparable. Sections could not be grouped so that comparisons of performance could be made to evaluate overlay thickness. Differences in overlay thickness always produced differences in total asphalt concrete thickness and, therefore, structural number which invalidates comparisons of performance.

Tables 4.2, 4.3, 4.4, 4.5 and 4.6 show performance data for SPS-5 pavements in Alabama, Alberta, Mississippi, Texas and California, respectively. Data from theses tables were used to plot Figures 4.1, 4.2, 4.3, 4.4, and 4.5 to examine the effects of overlay variables on smoothness degradation, rutting, fatigue cracking, longitudinal cracking, and transverse cracking, respectively.

Figure 4.1 indicates that, in California and Alberta, 1) virgin AC performs slightly better than AC with RAP in controlling loss of smoothness, and 2) thicker overlay might reduce the rate of smoothness degradation.

Figure 4.2 shows only that, in California, thicker overlays seem to rut less.

Figure 4.3 reveals that, in Alberta, California and Mississippi, 1) virgin AC is consistently better than AC with RAP in preventing fatigue cracking, and 2) fatigue cracking decreases as the overlay thickness increases.

Figure 4.4 shows that, in Alabama and California, 1) virgin AC and AC with RAP exhibit contradictory effects in preventing longitudinal cracking, and, 2) only in Alabama, does overlay thickness effect longitudinal cracking.

Figure 4.5 shows that virgin AC is better than AC with RAP in preventing transverse cracking, except for Alberta.

To get an overall view of the effects of milling and material type, distress indexes (DI) from Tables 4.2 to 4.6 are grouped and tabulated in Tables 4.7 and 4.8. A difference of at least 50 in DI was arbitrarily set as a requirement for determining a difference in performance of two sections. Data from Mississippi are not shown in Table 4.7 because total thicknesses were different for comparable sections. Data from California are not shown in Table 4.7 because all sections were milled.

Table 4.2 Performance Summary for Alabama SPS-5 Sections

Sections	Overlay AC (in)	Surface Prep.	Mix Type	M/O (in/in)	Total AC (in)	ΔIRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI*
0507	5.3	Mill2"	AC	2/5	7.3	0.15	8	0	6	0	29
0504	4.3	Min	AC	0/5	8.3	0.07	9	0	0	0	16
0509	3.2	Mill2"	RAP	2/2	5.2	0.09	6	4	90	8	117
0502	1.3	Min	RAP	0/2	5.3	0.07	6	40	140	58	251
0508	5.7	Mill2"	RAP	2/5	7.7	0.12	12	14	38	0	76
0503	4.1	Min	RAP	0/5	8.1	0.02	7	0	18	0	27
0506	3.0	Mill2"	AC	2/2	5.0	0.08	5	0	6	0	19
0505	1.4	Min	AC	0/2	5.4	0.07	5	18	140	24	194

1. Mill2" – Existing pavement surface milled to depth of 2 inches before overlay.
2. Min – Existing pavement surface not milled. Surface preparation includes patching only and/or crack sealing.
3. M/O – Design milling thickness / Design overlay thickness
4. DI – Distress index = 100 * ΔIRI + Rutting + FC + LC + TC

Table 4.3 Performance Summary for Alberta SPS-5 Sections

Sections	Overlay AC (in)	Surface Prep.	Mix Type	M/O (in/in)	Total AC (in)	ΔIRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
0507	6.6	Mill2"	AC	2/5	10.8	0.1	9	20	0	52	91
0504	4.8	Min	AC	0/5	11.3	0.3	18	20	24	38	130
0509	3.3	Mill2"	RAP	2/2	7.9	0.4	10	180	0	2	232
0502	2.1	Min	RAP	0/2	7.5	0.4	10	210	0	38	298
0508	7.0	Mill2"	RAP	2/5	10.6	0.2	7	40	58	24	149
0503	5.0	Min	RAP	0/5	11.4	0.2	13	160	0	26	219
0506	3.7	Mill2"	AC	2/2	8.3	0.1	12	110	0	44	176
0505	2.1	Min	AC	0/2	8.3	0.2	5	70	84	88	267

Table 4.4 Performance Summary for Mississippi SPS-5 Sections

Sections	Overlay AC (in)	Surface Prep.	Mix Type	M/O (in/in)	Total AC (in)	ΔIRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
0507	4.9	Mill2”	AC	2/5	5.9	0.1	27	0	40	2	79
0504	4.9	Min	AC	0/5	8.4	0.2	28	2	15	12	77
0509	2.3	Mill2”	RAP	2/2	2.7	0.0	18	150	0	64	232
0502	2.0	Min	RAP	0/2	5.9	0.3	24	140	170	145	509
0508	4.8	Mill2”	RAP	2/5	6.6	0.2	27	90	16	80	233
0503	4.6	Min	RAP	0/5	9.2	0.0	22	25	40	35	142
0506	1.8	Mill2”	AC	2/2	3.1	0.5	27	52	20	32	181
0505	2.0	Min	AC	0/2	6.4	0.0	17	20	30	30	117

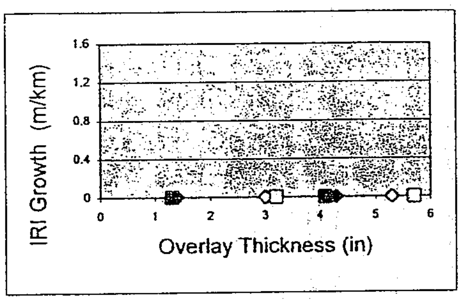
Table 4.5 Performance Summary for Texas SPS-5 Sections

Sections	Overlay AC (in)	Surface Prep.	Mix Type	M/O (in/in)	Total AC (in)	ΔIRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
0507	7.0	Mill2"	AC	2/5	14.8	0.0	13	0	1	0	14
0504	5.3	Min	AC	0/5	14	0.0	10	0	0	2	12
0509	4.3	Mill2"	RAP	2/2	12.1	0.0	8	0.3	16	40	64
0502	2.2	Min	RAP	0/2	11.4	0.1	10	0	2	55	77
0508	7.3	Mill2"	RAP	2/5	15.6	0.1	8	0	4	15	37
0503	5.3	Min	RAP	0/5	13.7	0.1	7	0	3	8	28
0506	3.9	Mill2"	AC	2/2	11.6	0.0	10	0	0	0	10
0505	2.0	Min	AC	0/2	11.4	0.2	10	0	0	40	76

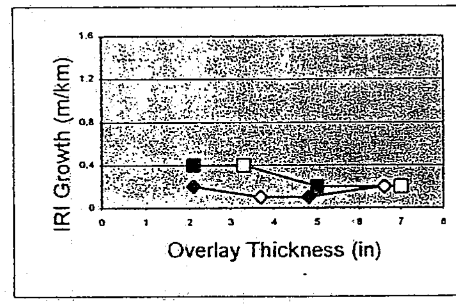
Table 4.6 Performance Summary for California SPS-5 Sections

Sections	Overlay AC (in)	Surface Prep.	Mix Type	B/S (in/in)	Total AC (in)	ΔIRI	Rutting (mm)	FC (m^2)	LC (m)	TC (No.)	DI
0507	6.7	Mill	AC	2/5	10.4	0.3	9	70	80	10	199
0504	5.7	Mill	AC	0/7	9.3	0.2	7	10	150	9	196
0509	4.4	Mill	RAP	2/2	8.3	1.5	9	240	2	110	511
0502	3.0	Mill	RAP	0/4	6.7	1.2	13	290	10	40	473
0508	6.6	Mill	RAP	2/5	10.7	0.0	6	10	25	80	121
0503	6.5	Mill	RAP	0/7	9.0	0.1	5	80	12	36	143
0506	4.3	Mill	AC	2/2	7.9	0.2	8	90	15	45	178
0505	3.6	Mill	AC	0/4	7.6	1.1	12	120	60	48	350

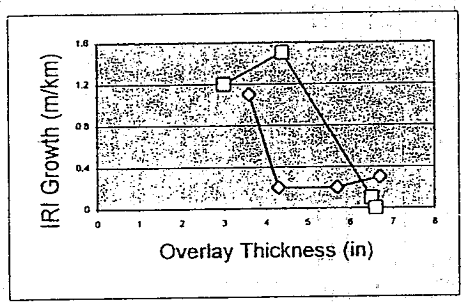
1. California SPS-5 sections were all milled and the differences among sections lie in overlay thickness and presence of binder layer.
2. B/S – Design binder thickness / Design surface thickness.



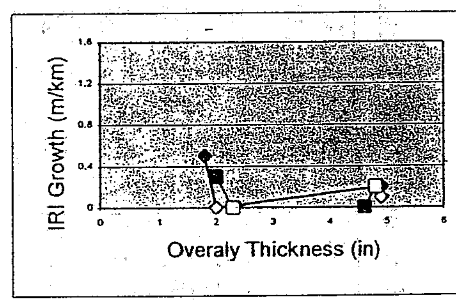
AL



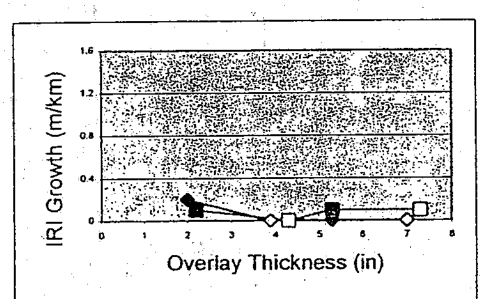
AB



CA



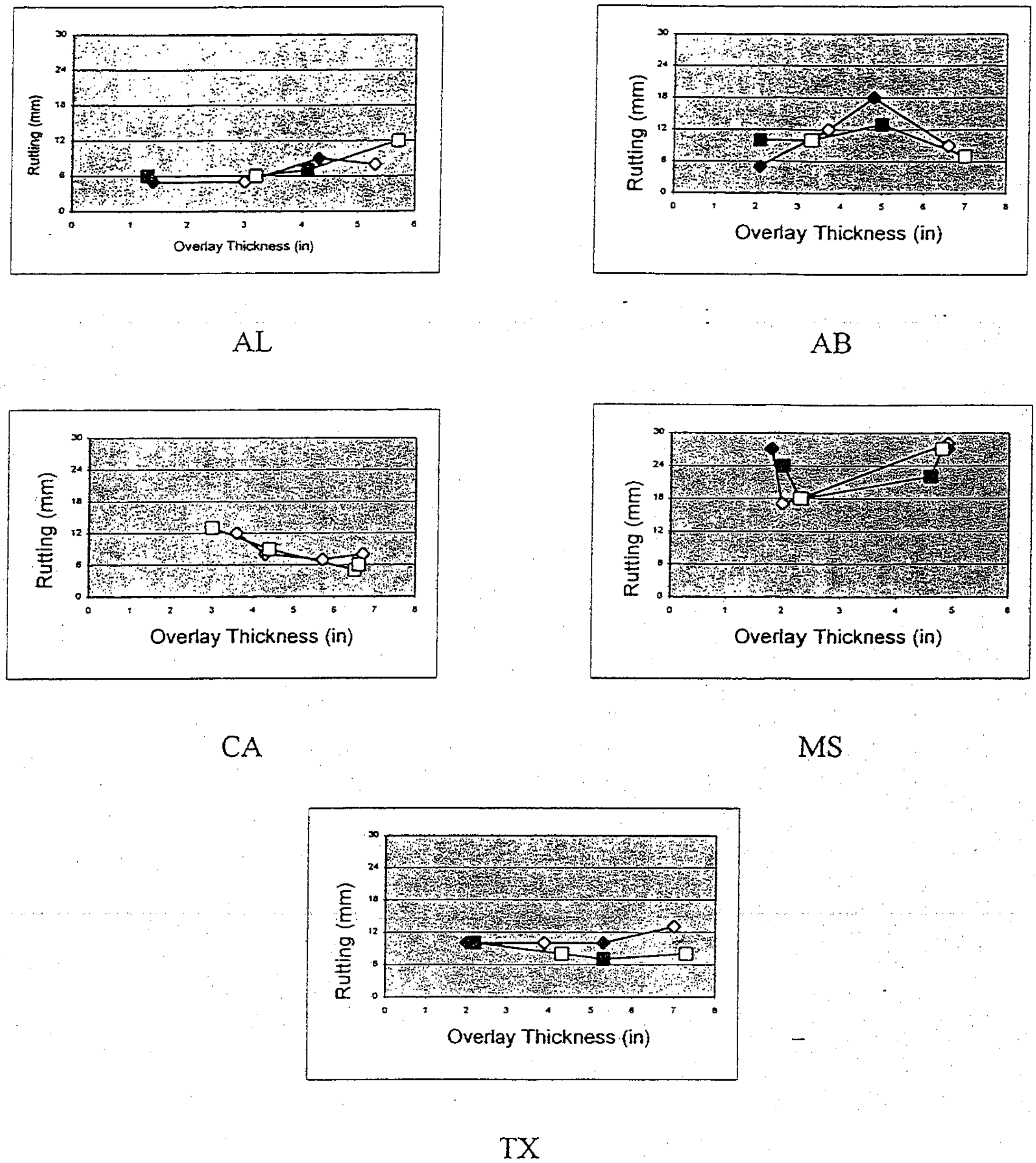
MS



TX

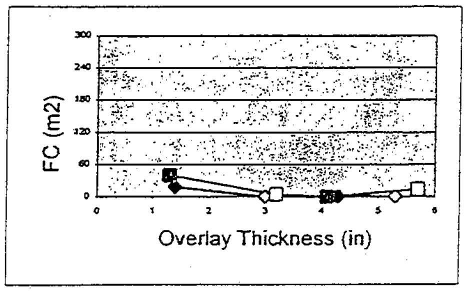
Black – AC light – RAP Blank – milled Filled – non-milled

Figure 4.1 Overlay Thickness versus Smoothness Degradation

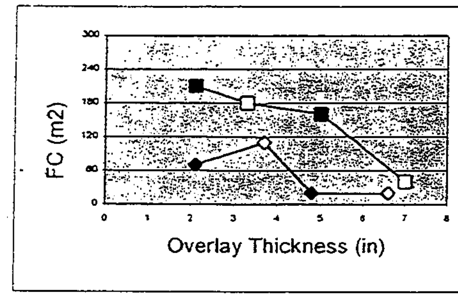


Black – AC   light – RAP   Blank – milled   Filled – non-milled

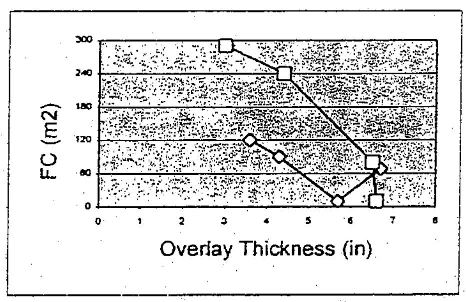
Figure 4.2 Overlay Thickness versus Rutting



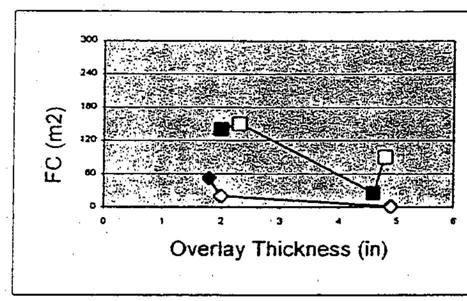
AL



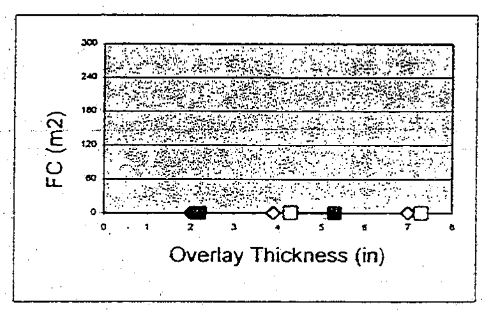
AB



CA



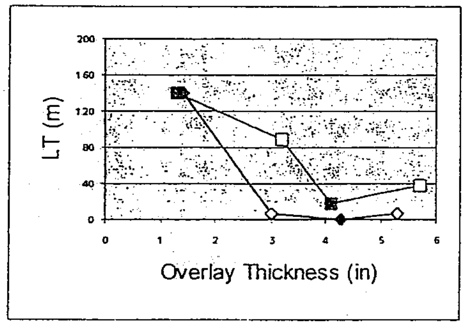
MS



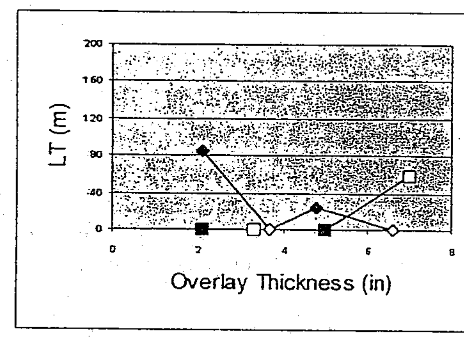
TX

Black – AC light – RAP Blank – milled Filled – non-milled

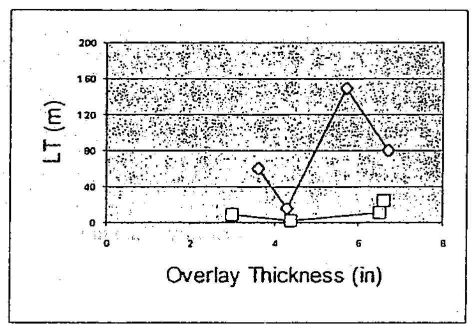
Figure 4.3 Overlay Thickness versus Fatigue Cracking



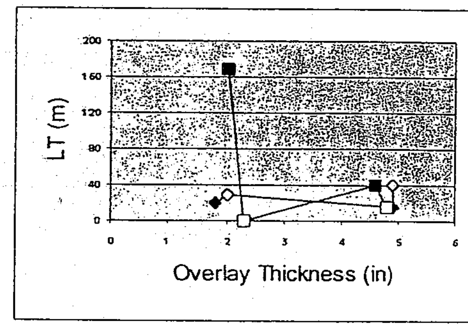
AL



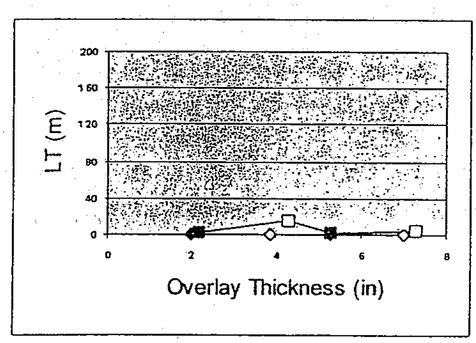
AB



CA



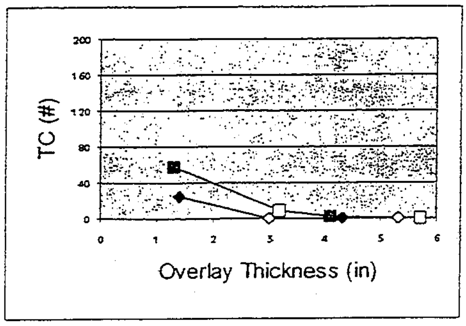
MS



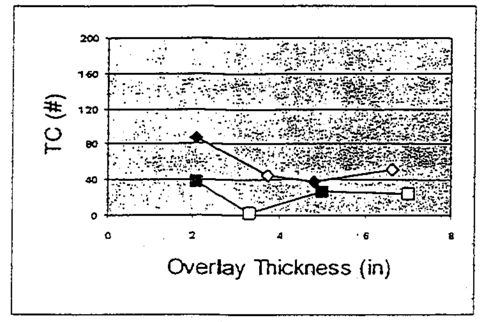
TX

Black – AC   light – RAP   Blank – milled   Filled – non-milled

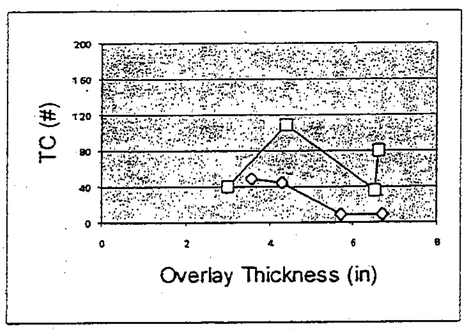
Figure 4.4 Overlay Thickness versus Longitudinal Cracking



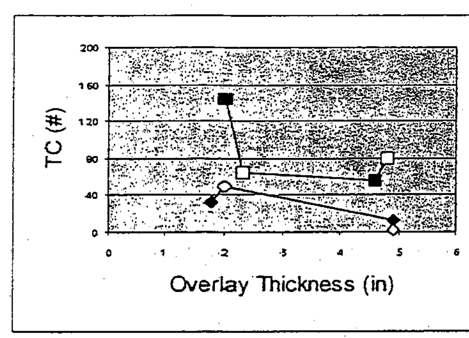
AL



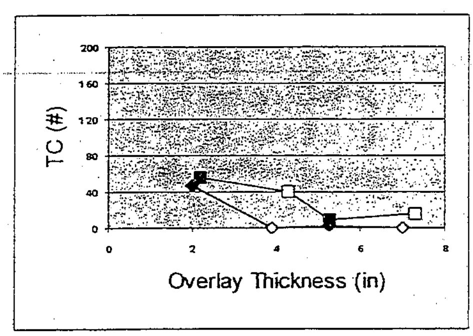
AB



CA



MS



TX

Black – AC light – RAP Blank – milled Filled – non-milled

Figure 4.5 Overlay Thickness versus Transverse Cracking

Table 4.7 reveals that milling of existing surface produces some benefits for thin overlay but for thick overlay the benefits are limited. Table 4.8 reveals that virgin mixes generally perform better than mixes with 30% RAP. The one notable exception is for 7 inch overlay in California where mixes with RAP perform somewhat better.

Table 4.7 Summarized Distress Indexes for Comparison of Milling Effects

State	**Virgin AC**				**AC with 30% RAP**			
M/O	**2/2**	**0/2**	**2/5**	**0/5**	**2/2**	**0/2**	**2/5**	**0/5**
AL	19	194	29	16	117	251	76	27
AB	176	267	91	130	232	298	149	219
TX	10	76	14	12	64	77	37	28
	Some benefits of milling indicated		No benefit of milling indicated		Some benefits of milling indicated in AL and AB		Some benefits of milling indicated in AB	

Table 4.8 Summarized Distress Indexes for Comparison of RAP Effects

State	Mill 2” OL (2/2)		Mill 5” OL (2/5)		No Mill 2” OL (0/2)		No Mill 5” OL (0/5)	
	Virgin	RAP	Virgin	RAP	Virgin	RAP	Virgin	RAP
AL	19	117	29	76	194	251	16	27
AB	176	232	91	149	267	298	130	219
MS	181	232	70	233	117	509	77	142
TX	10	64	14	37	76	77	12	28
	Virgin mixes better		Virgin mixes better in AB and MS		Virgin mixes better in AL and MS		Virgin mixes better in AB and MS	
CA	Mill 4” OL				Mill 7” OL			
	B/S = 2/2		B/S = 0/4		B/S = 2/5		B/S = 0/7	
	Virgin	RAP	Virgin	RAP	Virgin	RAP	Virgin	RAP
	178	511	350	473	199	121	196	143
	Virgin mixes better for 4” OL but RAP mixes better for 7” OL in CA							

CHAPTER 5:

CONCLUSIONS AND RECOMMENDATIONS

Conclusions and recommendations from this research can be summarized as follows:

Research revealed that, statistically speaking, the 1993 AASHTO flexible pavement design model provides biased prediction of the performance of flexible pavements with granular bases but unbiased prediction of performance of pavements with bound bases when using subgrade resilient moduli from laboratory testing. Serviceability degradation and subgrade resilient modulus, are key parameters for the 1993 AASHTO design model.

Analysis of SPS-1 sections indicated fatigue cracking as a serious distress manifestation for pavements with granular bases. "Full-depth" AC pavements performed better than other structural designs. Open graded drainable base layers improved the performance of pavements over fine-grained subgrades but pavements with dense graded black bases over coarse-grained subgrades performed better.

Analysis of SPS-5 sections indicated milling of existing pavement surfaces before overlay produces some benefits for thin overlay but no apparent benefit for thick overlay. Virgin AC performed better than AC with 30% RAP. Increased overlay thickness (thus total thickness) improved performance, especially for preventing fatigue and transverse cracking.

Data in the SHRP-LTPP database provided a good platform for conducting this research. But, it has deficiencies such as incompleteness of certain data and inconsistencies of some data elements which certainly diminishes confidence in the evaluations performed.

APPENDIX

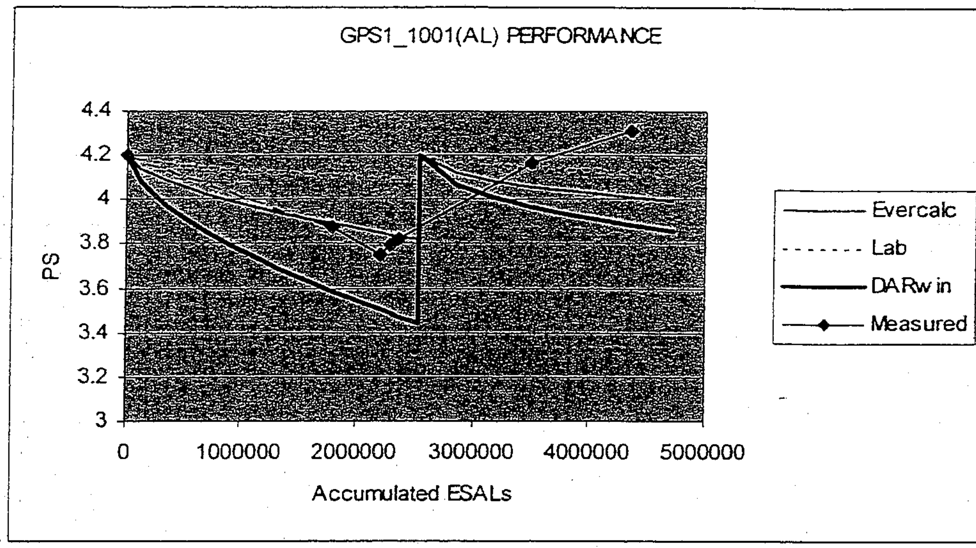
DATA USED IN AASHTO DESIGN EQUATION EVALUATION

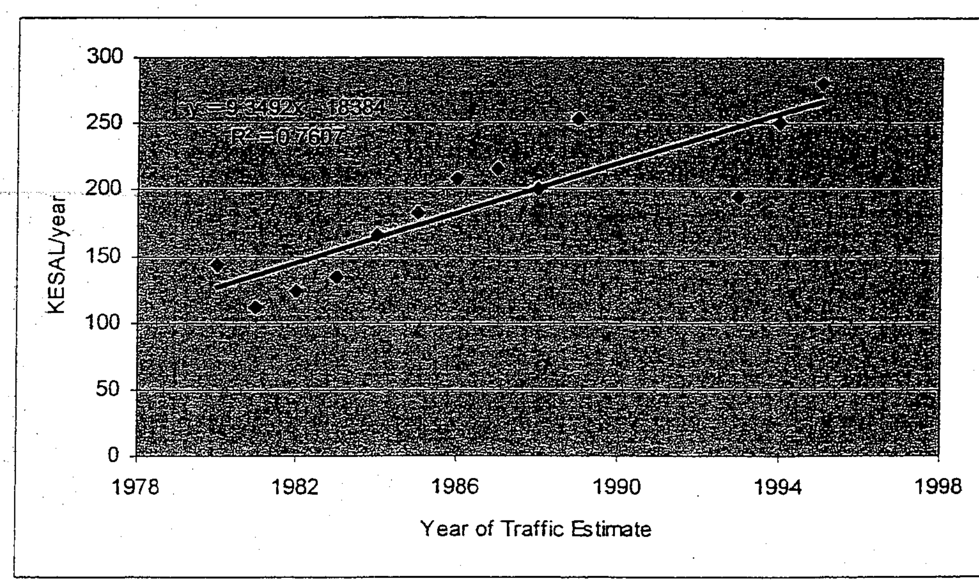
Modulus of Each Layer in Alabama GPS-1 Section

ID (AL)	CONDITION		AC (ksi)	BASE/SUBBASE (ksi)	SUBGRADE (psi)
1001	Evercalc	Wet	2030	23	5,075
		Dry	594	51	16,675
	Lab		-	24	26,390
	DARwin		-	-	6,670
4126	Evercalc	Wet	290	200	9,715
		Dry	800	108	9,425
	Lab		-	26	9,715
	DARwin		-	-	9,425
4127	Evercalc	Wet	-	-	-
		Dry	580	435	9,425
	Lab		-	21	13,340
	DARwin		-	-	8,990
4129	Evercalc	Wet	-	-	-
		Dry	732	101	7,105
	Lab		-	33/22	11,165
	DARwin		-	-	6,525
4155	Evercalc	Wet	580	145	12,325
		Dry	1918	290	32,625
	Experiment		-	33	17,690
	DARwin		-	-	15,950

Parameters for GPS1_1001(AL)

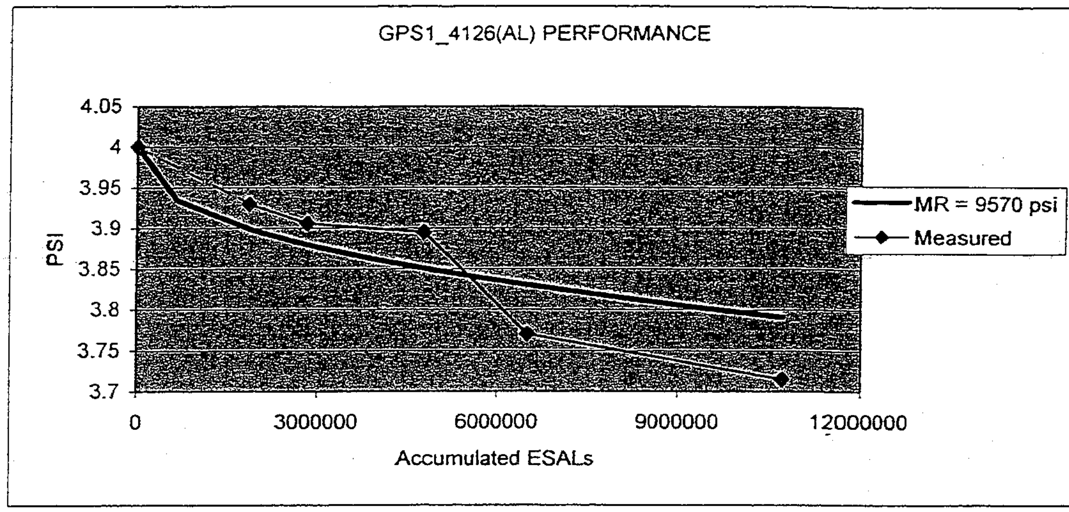
Parameters				Values
Construction Date				10/1/80
Overlay Date				6/1/93
SN (Original)	Layers	D_i(in)	a_i	4.48
	AC	3.3	0.44	
	GRB	6.2	0.15	
	GRSB	19.1	0.11	
SN (Overlayed)	Layers	D_i(in)	a_i	5.63
	AC	5.9	0.44	
	GRB	6.2	0.15	
	GRSB	19.1	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	10/1/80	0.58	4.2	0
	8/8/90	0.86	3.87	1776600
	4/3/92	0.97	3.75	2208100
	7/28/92	0.93	3.79	2295300
	10/22/92	0.9	3.83	2358700
	1/10/96	0.61	4.17	3504400
	1/27/98	0.5	4.31	4346800
M_R	Evercalc			10870 psi
	DARwin			6670 psi
	Lab			26390 psi

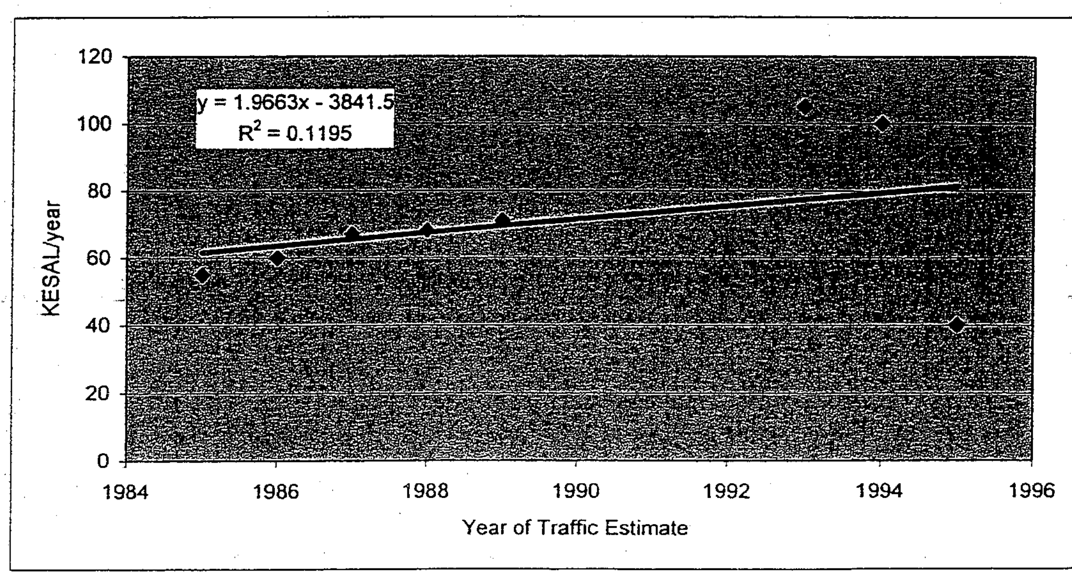

GPS1_1001(AL) PERFORMANCE
PS
4.4
4.2
4
3.8
3.6
3.4
3.2
3
0
1000000
2000000
3000000
4000000
5000000
Accumulated ESALs
Evercalc
Lab
DARwin
Measured


y = 9.3492x - 18384
R² = 0.7607
KESAL/year
300
250
200
150
100
50
0
1978
1982
1986
1990
1994
1998
Year of Traffic Estimate

Parameters for GPS1_4126(AL)

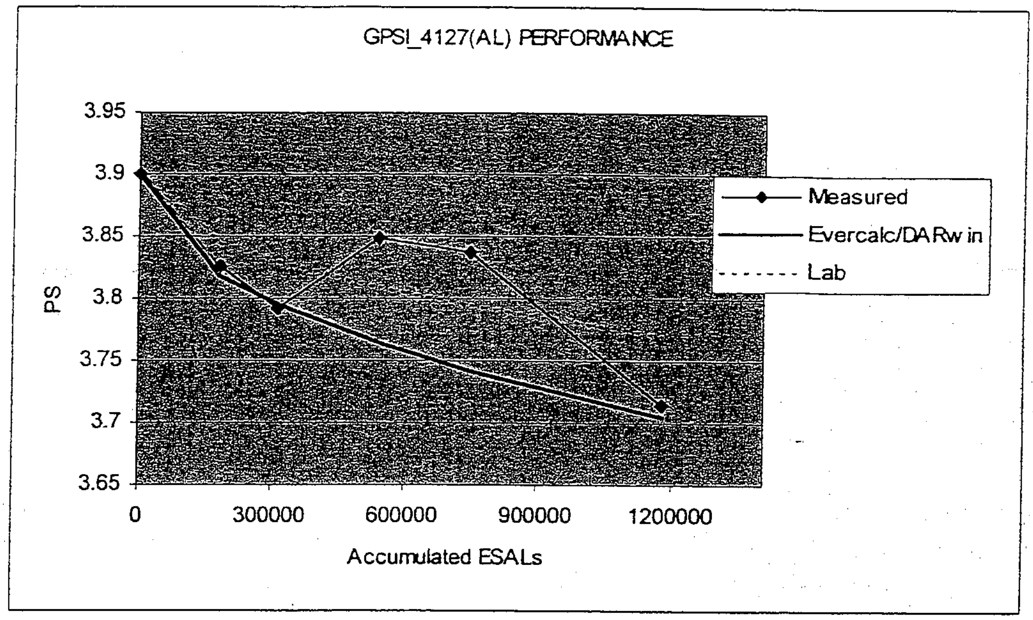
Parameters				Values
Construction Date				4/1/88
Overlay Date				N/A
SN	Layers	D_i(in)	a_i	7.79
	AC	13.1	0.44	
	GRSB	18.4	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	4/1/88	0.74	4.0	0
	12/6/90	0.81	3.93	1862000
	2/10/92	0.83	3.91	2821400
	2/2/94	0.84	3.89	4767700
	1/9/96	0.95	3.77	6514400
	5/11/99	1	3.71	10742000
M_R	Evercalc			9565 psi
	DARwin			9425 psi
	Lab			9715 psi

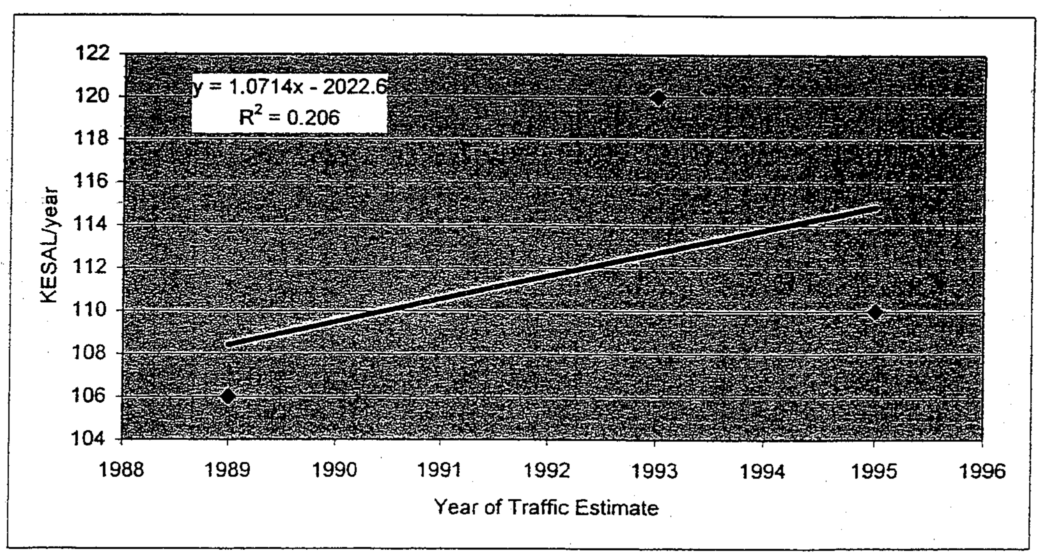

GPS1_4126(AL) PERFORMANCE
4.05
4
3.95
3.9
3.85
3.8
3.75
3.7
PSI
0
3000000
6000000
9000000
12000000
Accumulated ESALs
MR = 9570 psi
Measured


y = 1.9663x - 3841.5
R² = 0.1195
120
100
80
60
40
20
0
KESAL/year
1984
1986
1988
1990
1992
1994
1996
Year of Traffic Estimate

Parameters for GPS1_4127(AL)

Parameters				Values
Construction Date				8/1/74
Overlay Date				4/3/89
SN (Overlayed)	Layers	D_i(in)	a_i	5.51
	AC	10	0.44	
	GRB	7.4	0.15	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	4/3/89	0.87	3.9	0
	12/6/90	0.9	3.83	182800
	2/10/92	0.93	3.79	313500
	2/1/94	0.88	3.85	539500
	12/6/95	0.89	3.84	748500
	8/12/98	1	3.71	1177500
M_R	Evercalc			9425 psi
	DARwin			8990 psi
	Lab			13340 psi

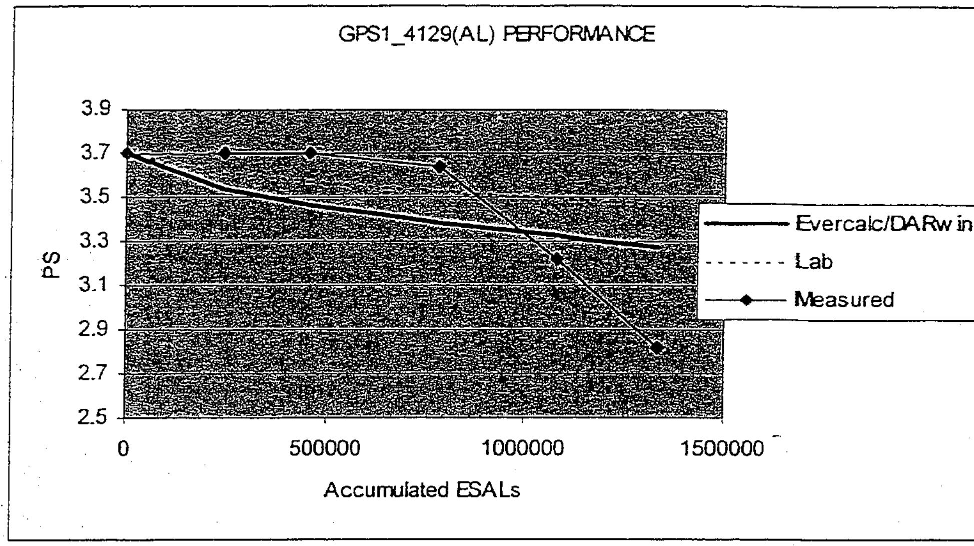

GPSI_4127(AL) PERFORMANCE
3.95
3.9
3.85
3.8
3.75
3.7
3.65
PS
0
300000
600000
900000
1200000
Accumulated ESALs
Measured
Evercalc/DARwin
Lab

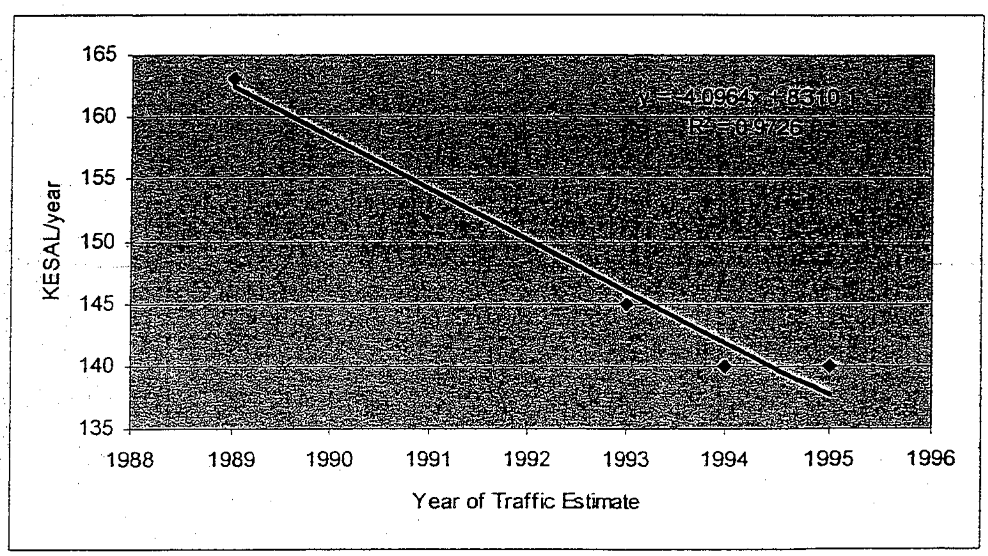

y = 1.0714x - 2022.6
R^2 = 0.206
122
120
118
116
114
112
110
108
106
104
KESAL/year
1988
1989
1990
1991
1992
1993
1994
1995
1996
Year of Traffic Estimate

Parameters for GPS1_4129(AL)

Parameters				Values
Construction Date				6/1/76
Overlay Date				6/1/89
SN	Layers	D_i(in)	a_i	4.53
	AC	4.5	0.44	
	GRB	12.6	0.15	
	GRSB	6	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs*
	6/1/89	1.01	3.7	0
	12/10/90	1.01	3.70	243000
	4/6/92	1.01	3.70	459000
	2/17/94	1.07	3.64	783000
	12/13/95	1.48	3.22	1080000
	9/3/97	1.93	2.82	1336500
M_R	Evercalc			7105 psi
	DARwin			6525 psi
	Lab			11165 psi

* Regression equation not used but assumed constant 162,000 ESALs/year.

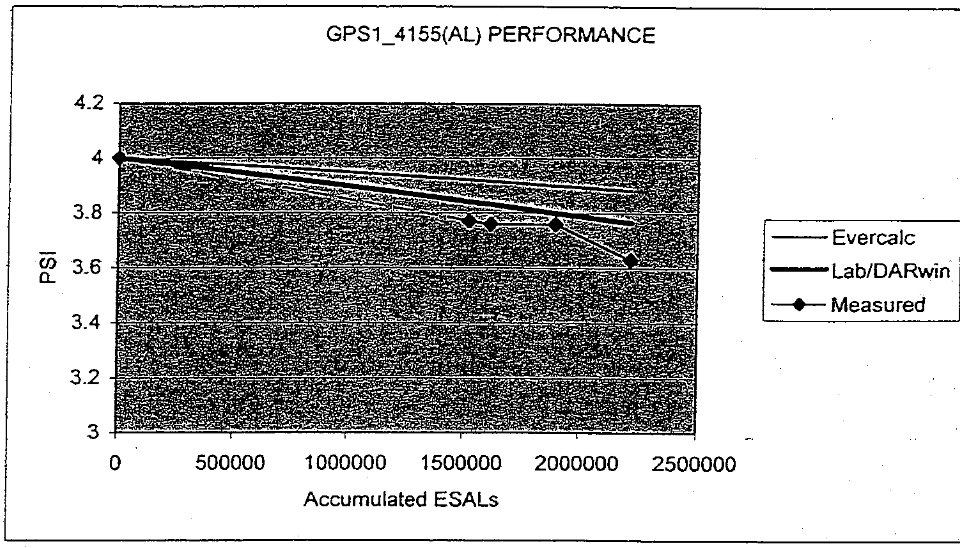

GPS1_4129(AL) PERFORMANCE
3.9
3.7
3.5
3.3
3.1
2.9
2.7
2.5
PS
0
500000
1000000
1500000
Accumulated ESALs
Evercalc/DARwin
Lab
Measured

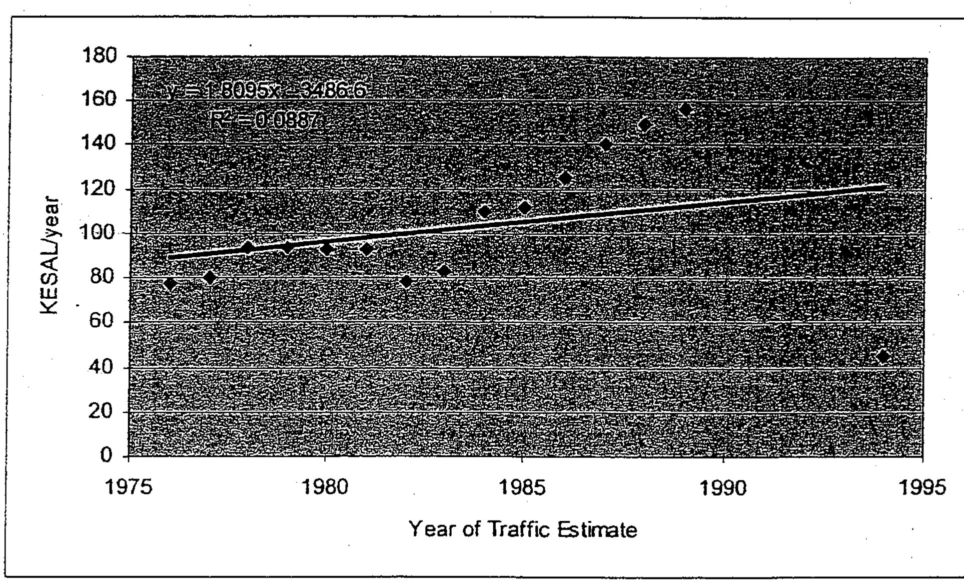

165
160
155
150
145
140
135
KESAL/year
1988
1989
1990
1991
1992
1993
1994
1995
1996
Year of Traffic Estimate
y = -4.0964x + 8310.1
R² = 0.9726

Parameters for GPS1_4155(AL)

Parameters				Values
Construction Date				10/1/75
Overlay Date				10/1/95
SN	Layers	D_i(in)	a_i	3.19
	AC	3.7	0.44	
	GRB	10.4	0.15	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	10/1/75	0.75*	4.0*	0
	6/8/90	0.95	3.77	1522700
	12/12/90	0.96	3.76	1616000
	8/24/92	0.96	3.76	1896000
	8/9/94	1.08	3.63	2216000
M_R	Evercalc			22463 psi
	DARwin			15950 psi
	Lab			17690 psi

* Regression equation not used to extrapolate IRI to 1975 to estimate the initial PSI. Visual extrapolation to estimate $P_i = 4.0$.


GPS1_4155(AL) PERFORMANCE
4.2
4
3.8
3.6
3.4
3.2
3
PSI
0
500000
1000000
1500000
2000000
2500000
Accumulated ESALs
Evercalc
Lab/DARwin
Measured

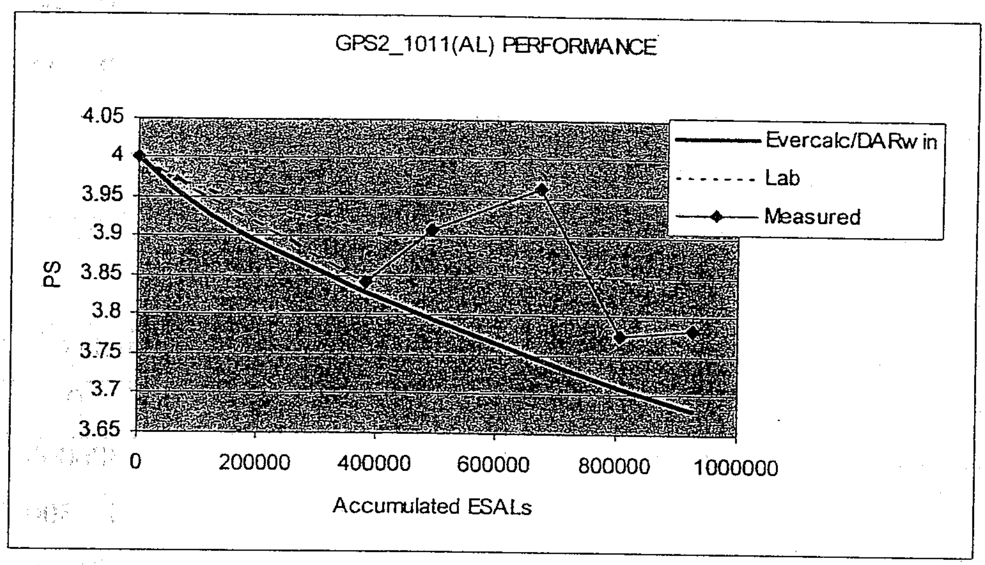

180
160
140
120
100
80
60
40
20
0
KESAL/year
y = 1.8095x - 3486.6
R² = 0.0887
1975
1980
1985
1990
1995
Year of Traffic Estimate

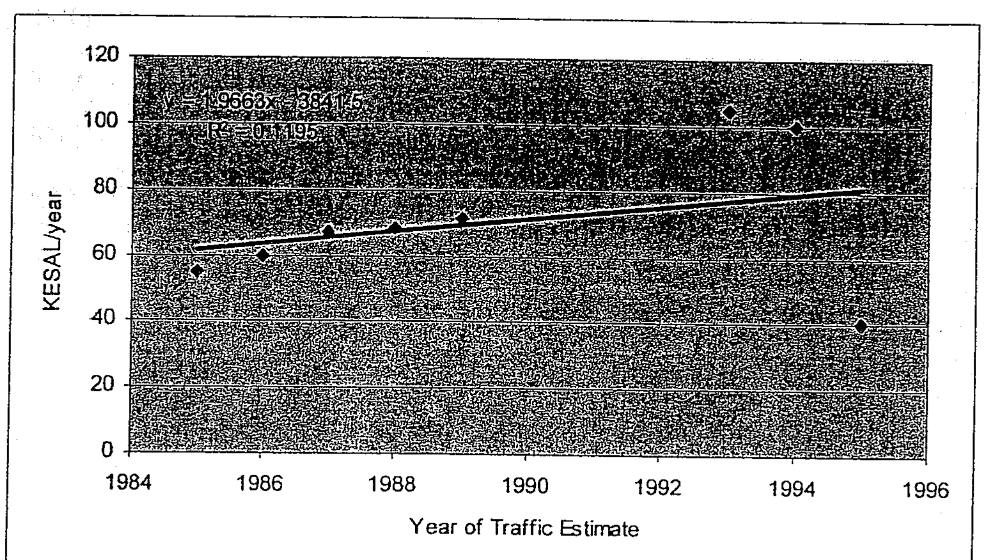
Modulus of Each Layer in Alabama GPS-2 Sections

ID (AL)	CONDITIONS		AC (ksi)	BASE/SUBBASE (ksi)	SUBGRADE (psi)
1011	Evercalc	Wet	166	414	7,975
		Dry	1020	380	8,990
	Lab		-	26/25	12,899
	DARwin		-	-	8,410
1019	Evercalc	Wet	431	509	6,670
		Dry	1160	509	7,250
	Lab		-	25	12,180
	DARwin		-	-	6,815
1021	Evercalc	Wet	758	95	11,600
		Dry	794	131	6,960
	Lab		-	26	10,585
	DARwin		-	-	7,250
4073	Evercalc	Wet	-	-	-
		Dry	1363	435	10,585
	Lab		-	23	11,890
	DARwin		-	-	9,860
4125	Evercalc	Wet	571	509	12,180
		Dry	1401	376	6,380
	Lab		-	23	13,050
	DARwin		-	-	6,960

Parameters for GPS2_1011(AL)

Parameters				Values
Construction Date				6/1/85
SN	Layers	D_i(in)	a_i	3.24
	AC	1.0	0.44	
	DGBB	5.0	0.34	
	GRSB	10.0	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	6/1/85	0.80	4.0	0
	12/6/90	0.90	3.84	386000
	2/11/92	0.84	3.91	491300
	2/1/94	0.79	3.96	671300
	8/16/95	0.96	3.78	806300
	8/12/98	0.95	3.78	926300
M_R	Evercalc			8478 psi
	DARwin			8410 psi
	Lab			12899 psi

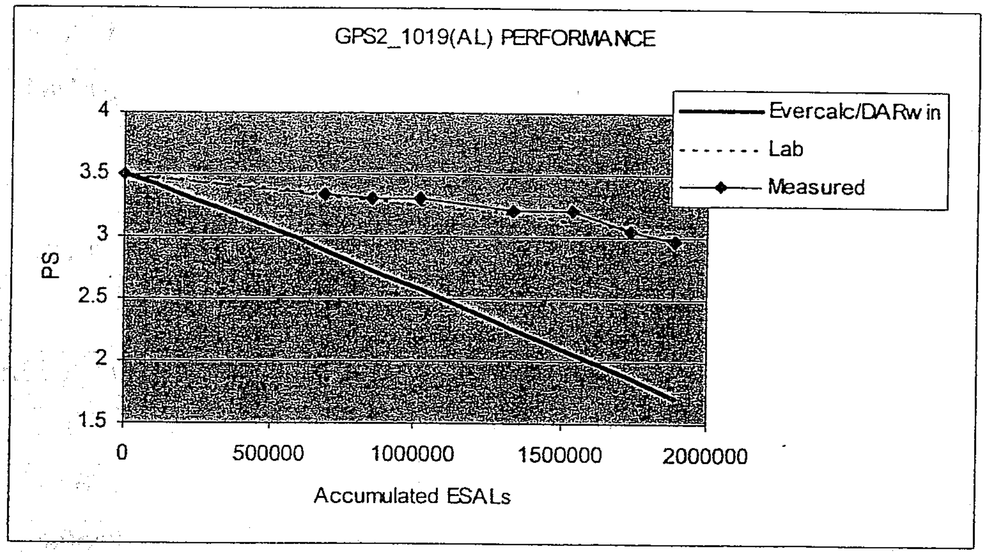

GPS2_1011(AL) PERFORMANCE
Evercalc/DARWin
Lab
Measured
PS
4.05
4
3.95
3.9
3.85
3.8
3.75
3.7
3.65
0
200000
400000
600000
800000
1000000
Accumulated ESALs

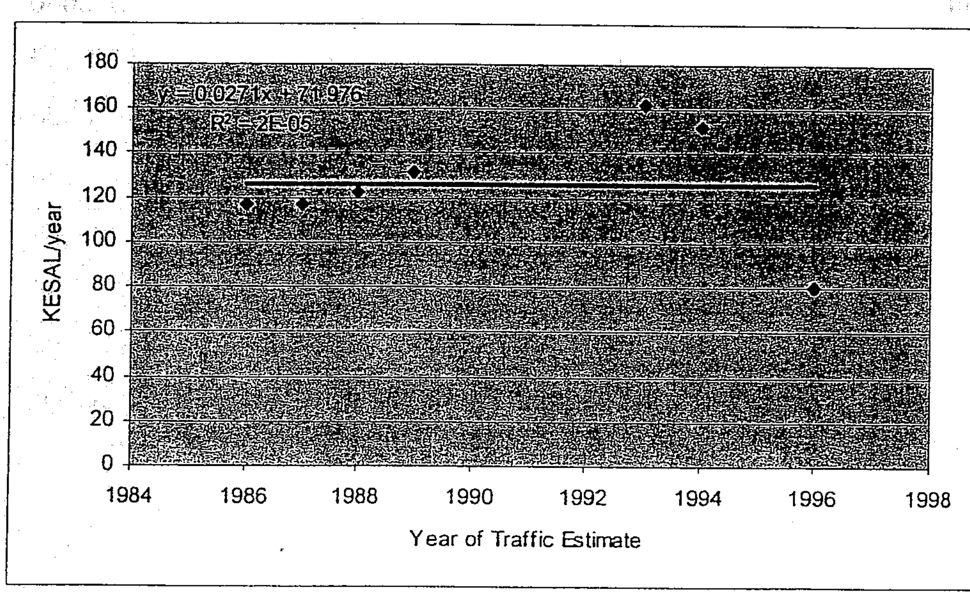

y = 1.9663x - 3841.5
R² = 0.1195
KESAL/year
120
100
80
60
40
20
0
1984
1986
1988
1990
1992
1994
1996
Year of Traffic Estimate

Parameters for GPS2_1019(AL)

Parameters				Values
Construction Date				10/1/84
SN	Layers	D_i(in)	a_i	3.17
	AC	3.5	0.44	
	DGBB	3	0.34	
	GRSB	5.5	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	10/1/84	1.2*	3.5*	0
	6/5/90	1.37	3.35	690000
	7/11/91	1.42	3.30	855000
	8/21/92	1.41	3.31	1020000
	8/8/94	1.51	3.21	1340000
	12/21/95	1.51	3.21	1540000
	4/21/97	1.70	3.04	1740000
	4/22/98	1.79	2.96	1890000
M_R	Evercalc			6960 psi
	DARwin			6815 psi
	Lab			12180 psi

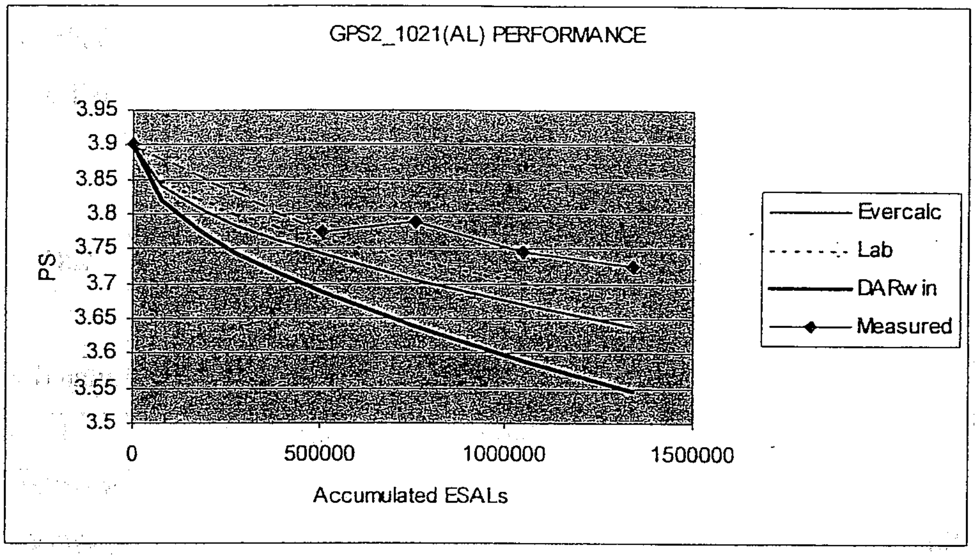
* Regression equation not used to estimate initial PSI. Visual extrapolation based on first five points used to estimate initial IRI for computing initial PSI.

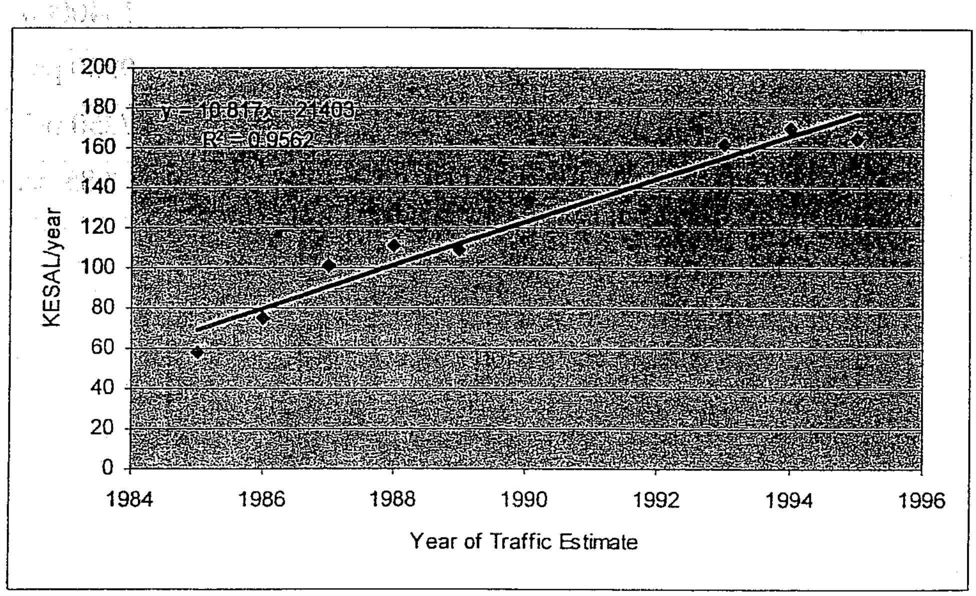

GPS2_1019(AL) PERFORMANCE
Evercalc/DARwin
Lab
Measured
PS
4
3.5
3
2.5
2
1.5
0
500000
1000000
1500000
2000000
Accumulated ESALs


y = 0.0271x + 71.976
R² = 2E-05
KESAL/year
180
160
140
120
100
80
60
40
20
0
1984
1986
1988
1990
1992
1994
1996
1998
Year of Traffic Estimate

Parameters for GPS2_1021(AL)

Parameters				**Values**
Construction Date				6/1/85
SN	Layers	D_i(in)	a_i	4.81
	AC	3.1	0.44	
	GDBB	4.5	0.34	
	GRSB	17.4	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	6/1/85	0.89	3.9	0
	6/6/90	0.96	3.77	505000
	4/3/92	0.95	3.79	757000
	2/17/94	0.99	3.74	1047000
	12/13/95	1.01	3.73	1340000
M_R	Evercalc			9280 psi
	DARwin			7250 psi
	Lab			10585 psi

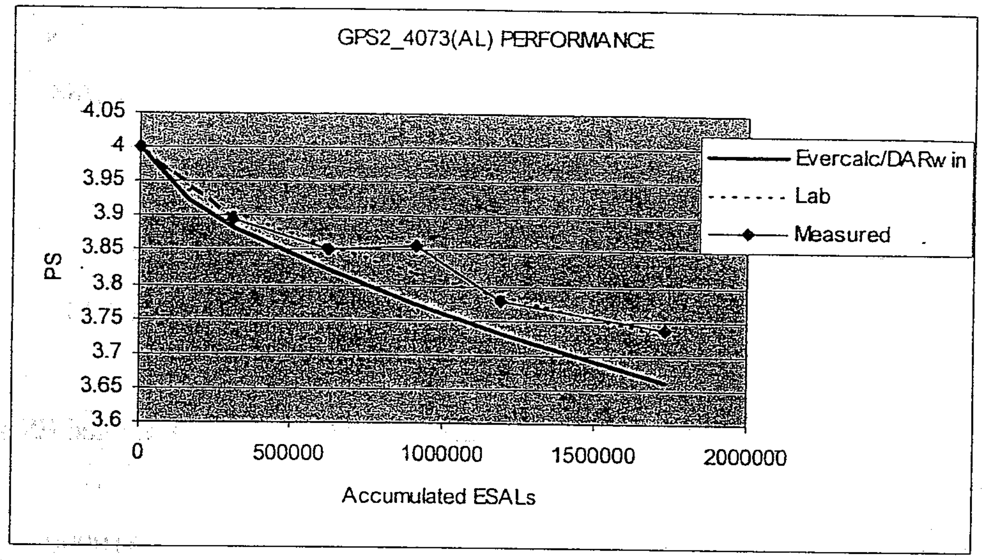

GPS2_1021(AL) PERFORMANCE
PS
3.95
3.9
3.85
3.8
3.75
3.7
3.65
3.6
3.55
3.5
0
500000
1000000
1500000
Accumulated ESALs
Evercalc
Lab
DARwin
Measured


KESAL/year
200
180
160
140
120
100
80
60
40
20
0
y = 10.817x - 21403
R² = 0.9562
1984
1986
1988
1990
1992
1994
1996
Year of Traffic Estimate

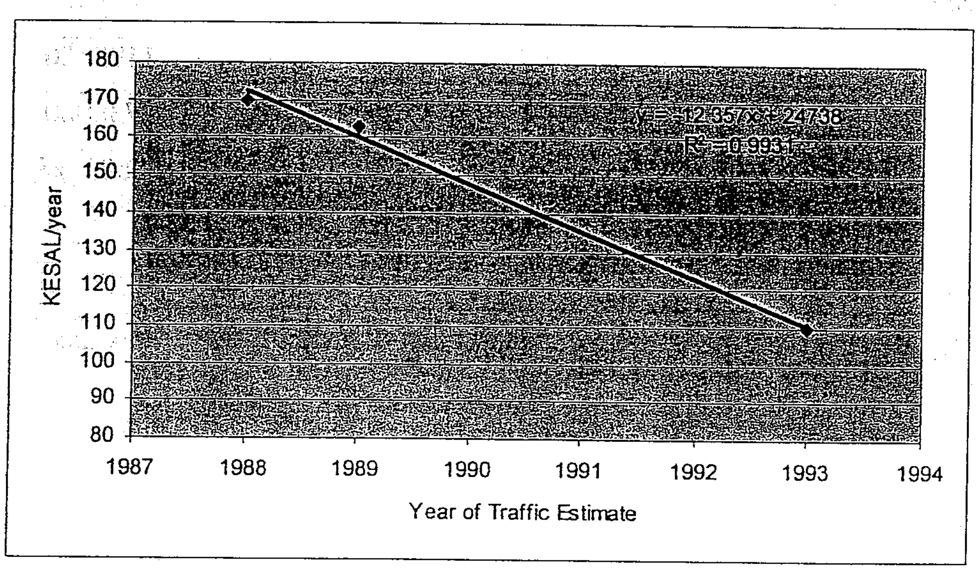
Parameters for GPS2_4073(AL)

<table>
<tr><th colspan="4">Parameters</th><th>Values</th></tr>
<tr><td colspan="4">Construction Date</td><td>6/1/88</td></tr>
<tr><td rowspan="5">SN</td><td>Layers</td><td>D_i(in)</td><td>a_i</td><td rowspan="5">4.14</td></tr>
<tr><td>AC</td><td>1.5</td><td>0.44</td></tr>
<tr><td>DGBB</td><td>7.1</td><td>0.34</td></tr>
<tr><td>GRSB</td><td>5.2</td><td>0.11</td></tr>
<tr><td>LTS</td><td>4.9</td><td>0.10</td></tr>
<tr><td rowspan="7">Traffic And Smoothness</td><td>Date</td><td>IRI(m/km)</td><td>PSI</td><td>Accumulated ESALs*</td></tr>
<tr><td>6/1/88</td><td>0.82</td><td>4.0</td><td>0</td></tr>
<tr><td>5/8/90</td><td>0.85</td><td>3.90</td><td>310000</td></tr>
<tr><td>4/15/92</td><td>0.89</td><td>3.85</td><td>620000</td></tr>
<tr><td>2/22/94</td><td>0.89</td><td>3.86</td><td>913000</td></tr>
<tr><td>12/7/95</td><td>0.96</td><td>3.78</td><td>1193000</td></tr>
<tr><td>4/14/99</td><td>1.00</td><td>3.74</td><td>1730000</td></tr>
<tr><td rowspan="3">M_R</td><td colspan="3">Evercalc</td><td>10585 psi</td></tr>
<tr><td colspan="3">DARwin</td><td>9860 psi</td></tr>
<tr><td colspan="3">Lab</td><td>11890 psi</td></tr>
</table>

*Regression equation not used to estimate accumulated ESALs. Assumed 160,000 ESALs/year for test period.



GPS2_4073(AL) PERFORMANCE
PS
4.05
4
3.95
3.9
3.85
3.8
3.75
3.7
3.65
3.6
0
500000
1000000
1500000
2000000
Accumulated ESALs
Evercalc/DARwin
Lab
Measured

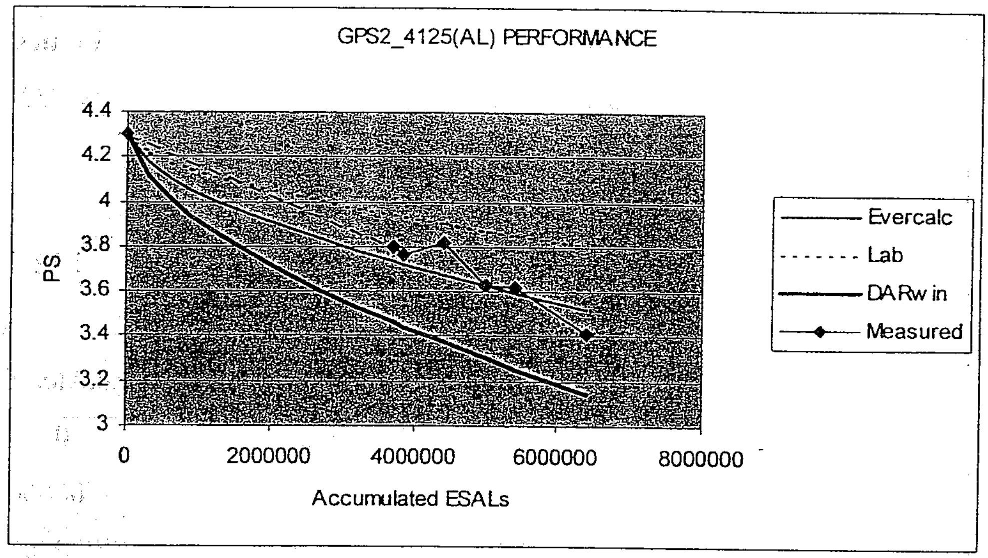


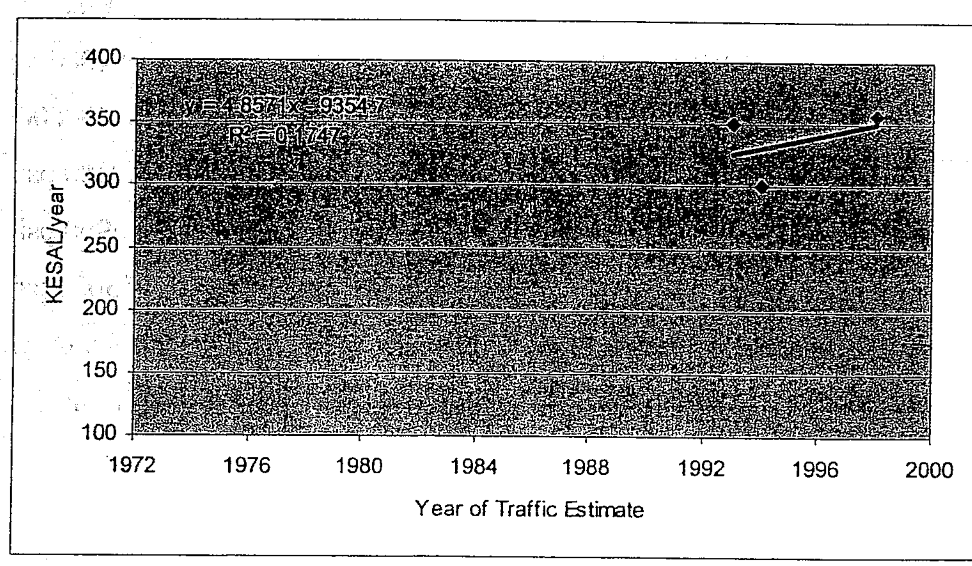
KESAL/year
180
170
160
150
140
130
120
110
100
90
80
1987
1988
1989
1990
1991
1992
1993
1994
Year of Traffic Estimate
y = -12.357x + 24738
R² = 0.9931

Parameters for GPS2_4125(AL)

Parameters				**Values**
Construction Date				6/1/72
SN	Layers	D_i(in)	a_i	4.32
	AC	3.4	0.44	
	DGBB	6.2	0.34	
	GRSB	6.5	0.11	
Traffic And Smoothness	Date	IRI(m/km)	PSI	Accumulated ESALs
	6/1/72	0.50*	4.3*	0
	6/7/90	0.94	3.80	3700000
	12/11/90	0.97	3.76	3850000
	7/29/92	0.93	3.81	4400000
	8/23/94	1.10	3.63	5000000
	12/19/95	1.10	3.62	5400000
	4/20/99	1.30	3.42	6400000
M_R	Evercalc			9280 psi
	DARwin			6960 psi
	Lab			13050 psi

* Regression equation was not used to estimate initial PSI. Extrapolation produced $P_i \approx 4.7$ which was considered too high. Minimum IRI = 0.5 m/km assumed.


GPS2_4125(AL) PERFORMANCE
PS
4.4
4.2
4
3.8
3.6
3.4
3.2
3
0
2000000
4000000
6000000
8000000
Accumulated ESALs
Evercalc
Lab
DARw in
Measured


y = 4.8571x - 9354.7
R² = 0.1747
KESAL/year
400
350
300
250
200
150
100
1972
1976
1980
1984
1988
1992
1996
2000
Year of Traffic Estimate