SHRP-P-680

Early Analyses of Long-Term Pavement Performance General Pavement Studies Data

Lessons Learned and Recommendations for Future Analyses

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Abstract

The purpose of this volume is to share the experience gained and lessons learned by the research staff during these early data analyses, and to recommend procedures for future analysts. In order to share the experience, a review is provided of the techniques used (and reported in detail in Volumes 3 and 4). Shortcomings of the LTPP Data Base available for the early analyses are discussed, and the data base expectations for future analyses were identified. Some interesting and useful distress and roughness prediction models were developed that show the effects of several design variables.

During the course of the research, a number of other analytical procedures for developing predictive equations were identified that might be of use in future analyses. These are identified and described briefly. Ten techniques used by the research staff for evaluating the AASHTO design equations are identified, and recommendations for future evaluations of design equations are provided.

Executive Summary

These early analyses on data from the General Pavement Studies (GPS) were considered by many as primarily a means to sort out the data base in terms of its limitations and potential uses, and as pilot studies to aid in selection of analytical techniques for future analyses when more time-sequence data are available. The objectives of Volume 5 are to document the experience gained from these early analyses and to offer suggestions on the basis of this experience to future analysts. This volume includes discussion of:

- data limitations and recommended actions to improve the data base and/or deal with limitations,
- data expectations for future analyses,
- procedures that did not work and recommended procedures for the future, and
- potential modeling and analysis techniques for future use.

Before proceeding with discussions of future activities, it is worthwhile to identify the products now available from these early analyses:

- usable data bases for combinations of LTPP pavement and distress types,
- statistical characterizations of the data,
- identification of biases in the data,
- interim distress and roughness models for several pavement types/designs,
- valuable insight as to the need for regional models,
- procedures for developing models from LTPP data,
- identification of variables having significant impacts on specific distresses, with implications for improvement of design,
- procedures for conducting sensitivity analyses on LTPP data,

- identification of procedures that don't work,

- identification of mechanistic variables and "clusters" for future modeling,

- identification of the very serious shortcomings of the AASHTO design equations,

- identification of potential improvements to the AASHTO design equations, and

- recommendations to follow in future analyses.

These products clearly demonstrate the potential value of the LTPP data base. Based on the experience of the research staff, future analytical objectives should include:

- development of distress and roughness models for use in design procedures, pavement management and sensitivity analyses,

- evaluation of State Highway Agency design procedures using LTPP data,

- calibration of existing mechanistic-empirical models using LTPP data,

- combining knowledge from SHRP studies of asphalt, concrete and long-term performance to improve performance models and gain additional insight into effects of independent variables on performance,

- development of models for layer stiffnesses in terms of material properties,

- follow up on unexpected phenomena resulting from analyses; and

- evaluation of seasonal changes in layer stiffnesses and surface profiles.

The details of the sensitivity analyses and the evaluations of the AASHTO design equations appear in Volumes 3 and 4, respectively. They also appear in summary form in Volume 5. The procedures developed and reported for conducting sensitivity analyses appeared to work very well. However, future study may result in better techniques for developing the required predictive equations and for evaluating the relative sensitivities of dependent variables (distresses or performance measures) to variations in the independent variables (such as layer thicknesses, material properties, environment, and traffic loadings).

It is important to understand that LTPP is a long-term study, and that these early data analyses have been constrained by lack of time-sequence data that can only be derived from long-term monitoring of the test sections. Also, a great number of the test sections had not yet experienced pavement distresses, so a great many of the test sections could not be included in the sensitivity analyses. For example, it was not possible to study alligator fatigue cracking in flexible pavements, because there were only 18 test sections reported to have any medium or high severity alligator cracking. Traffic was represented by estimates by the State Highway Agencies of cumulative ESALs up to the time that monitoring of the test sections began. Also, initial roughness (International Roughness Index, IRI) had to be derived from State Highway Agency estimates of initial Present Serviceability Indices (PSIs). Other limitations are discussed in Volumes 2, 3, and 4.

The lack of time sequence data did not limit the evaluations of the AASHTO design equations, as they involved direct comparisons of the predicted and estimated ESALs to produce the observed loss of PSI. These comparisons clearly indicated that both the flexible and rigid pavement design equations are poor prediction equations and that their use will frequently lead to unconservative designs.

Although the data available for future analyses will be greatly improved, some of the data limitations will not change. These include:

- initial PSI and IRI estimates (other than newer pavements where backcasting may be accomplished from time-series IRI data),

- missing inventory data,

- estimates of cumulative ESALs prior to monitoring,

- lack of initial stiffness values for HMAC test sections,

- pavement condition prior to overlay for GPS-6A, GPS-7A, and GPS-9 test sections, and

- lack of percent voids in mineral aggregate (VMA) for HMAC.

There will also be, however, some important data base improvements for future analyses, which will include:

- time-sequence distress data,

- time-sequence traffic data,

- resilient moduli for materials (subgrade, base, subbase, and HMAC. Elastic moduli for PCC layers are already available),

- seasonal effects from the seasonal monitoring program,

- more test sections displaying distress,

- more precise distress measurements from increased resolution for distress reductions from photographic negatives,

- radar measurements of layer thicknesses (FHWA is initially planning to measure 100 GPS test sections), and

- more precise data as a result of thorough QA/QC checks.

- backcalculated layer moduli for flexible pavements (already accomplished for rigid pavements).

It should be noted that the data limitations discussed above for the GPS data will generally not apply for the Specific Pavement Studies (SPS) data. The data from these constructed projects will be of much higher quality and be much more thorough than those from the existing GPS projects.

While the analyses were in progress, the research staff received a number of suggestions for alternative approaches to developing predictive equations. These are identified below:

- discriminant analysis

- probabilistic failure-time models to include "unobserved distress data"

- survival analysis

- Neural Network Approach

- Bayesian analysis

- nonlinear regression analysis

- advanced modern regression techniques

- mechanistic based models

Procedures for future evaluations of design procedures were recommended, using the experience from the evaluations of the AASHTO design equations as a basis. The primary technique recommended is direct comparison of predicted values to actual values. This can be predicted ESALs to cause a measured level of deterioration versus the actual ESALs, as in these evaluations of the AASHTO design equations, or comparisons of predicted to actual levels of distress (or deterioration). For overlays, the overlay thickness required by a design procedure can be compared to actual thicknesses and their performance.

The comparisons discussed above were followed by a variety of other procedures to learn more about the causes of differences between predicted and measured performance. Eight such procedures were described and recommended for future use as appropriate. The choice of procedures employed should depend on the nature of the predictive equation itself and on the results of the comparisons of predicted to actual performance.

Overall, these limited early results clearly demonstrate the potential power and usefulness of the LTPP data base. This is only a small beginning toward realizing the many possibilities.

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Introduction

This report, due to the diversity of the research works and the bulk of the text required to describe them, has been produced in four volumes and an Executive Summary. The overall report is entitled "Early Analyses of LTPP General Pavement Studies Data", but each of the volumes has an additional title as follows:

Volume 1 - "Executive Summary"
Volume 2 - "Data Processing and Evaluation"
Volume 3 - "Sensitivity Analyses for Selected Pavement Distresses"
Volume 4 - "Evaluation of the AASHTO Design Equations and Recommended Improvements"
Volume 5 - "Lessons Learned and Recommendations for Future Analyses of LTPP Data"

Each volume is written as a "stand alone" document, but it will be useful to refer to other volumes for additional detail.

This document (Volume 5) describes the lessons learned from the experience of conducting the analyses reported in Volumes 3 and 4, and provides recommendations for future analyses of LTPP Data. This work was supported by SHRP Contract P-020, "Data Analysis", which served as the primary vehicle for harvesting the results from the first five years of the SHRP Long-Term Pavement Performance (LTPP) studies and transforming this new information into implementable products supporting the LTPP goal and objectives. The research was conducted by Brent Rauhut Engineering Inc. and ERES Consultants, Inc.

The goal for the LTPP Studies, as stated in "Strategic Highway Research Plans", May 1986, is:

> "To increase pavement life by investigation of various designs of pavement structures and rehabilitated pavement structures, using different materials and under different loads, environments, subgrade soil and maintenance practices."

LTPP Objectives and Expected Products

The following six objectives were established by the SHRP Pavement Performance Advisory Committee in 1985 to contribute to accomplishment of the overall goal:

1. Evaluate existing design methods.

2. Develop improved design methods and strategies for pavement rehabilitation.

3. Develop improved design equations for new and reconstructed pavements.

4. Determine the effects of (1) loading, (2) environment, (3) material properties and variability, (4) construction quality, and (5) maintenance levels on pavement distress and performance.

5. Determine the effects of specific design features on pavement performance.

6. Establish a national long-term pavement data base to support SHRP objectives and future needs.

This research was the first to utilize the National Pavement Data Base (later renamed the "LTPP Data Base" and sometimes called the "LTPP Information Management System) to pursue these objectives. The early products that were expected from this data analysis are listed below and related to project tasks (to be described later):

1. A better understanding of the effects of a broad range of loading, design, environmental, materials, construction and maintenance variables on pavement performance (Task 2).

2. Evaluation of and improvements to the models included in the 1986 AASHTO Pavement Design Guide (Tasks 3 and 4).

3. Evaluation and Improvement of AASHTO overlay design procedures using data from the General Pavement Studies (GPS) (Task 5).

4. Data analysis plans for future analyses as time-sequence data for the GPS and Specific Pavement Studies (SPS) data enter the National Pavement Data Base (NPDB) and the National Traffic Data Base (NTDB) to offer opportunities for further insight and design improvements (Task 6).

This project began with development of tentative analysis plans for this initial analytical effort. These plans were presented July 31, 1990 to the Expert Task Group on Experimental Design and Analysis, and on August 2, 1990 to the highway community in a SHRP Data Analysis workshop. A detailed work plan was developed from the initial plans, in consideration of comments and guidance received from these and subsequent meetings. Guidance was furnished to the contractors throughout the research by a Data Analysis Working Group (Composed of SHRP Staff and SHRP Contractors), the Expert Task Group on Experiment Design and Analysis, and the Pavement Performance Advisory Committee.

Research Tasks

The specified tasks for SHRP Contract P-020a were:

Task 1	-	Data Evaluation Procedure and Workshop
Task 1A	-	Data Processing and Evaluation
Task 2	-	Sensitivity Analysis of Explanatory Variables in the National Pavement Performance Data Base
Task 3	-	Evaluation of the AASHTO Design Equations
Task 4	-	Improvement of the AASHTO Design Equations
Task 5	-	Evaluate and Improve AASHTO Overlay Procedures Using GPS Data
Task 6	-	Future LTPP Data Analysis Plans

The relationships between the tasks and the general flow of the research appear in Figure 1.1. The task documented in this volume is Task 6. This volume documents the experiences gained from these early data analyses and recommendations for future analyses, which incorporate this experience and data expectations as monitoring of test sections continues over time.

Data Bases Used in the Analyses

The LTPP Data Base will eventually include data for both General Pavement Studies (GPS) and Specific Pavement Studies (SPS), but only the GPS data was even marginally adequate for these early analyses. At the time these analyses were conducted, the SPS data were only beginning to be entered into the IMS for projects recently constructed, and most of the projects are not yet constructed. It should be noted that all of the data collected for LTPP studies are for test sections 500 feet (152.4 meters) in length and the width of the outside traffic lane. All test sections are located in the outside traffic lane.

The GPS experiments are identified and briefly described in Table 1.1. Except for Task 5, only the data for pavements that had not yet been rehabilitated; e.g., were in their first service period before being overlaid or otherwise rehabilitated (GPS-1 through GPS-5), were used in the analyses. The limited data bases available for the pavements with overlays were used for Task 5, "Evaluate and Improve AASHTO Overlay Procedures Using GPS Data" (see Volume 4 of this report). There were not sufficient test sections in GPS-6, GPS-7, and GPS-9, for which condition prior to overlay was known, to support development of reasonable predictive models for conducting sensitivity analyses.

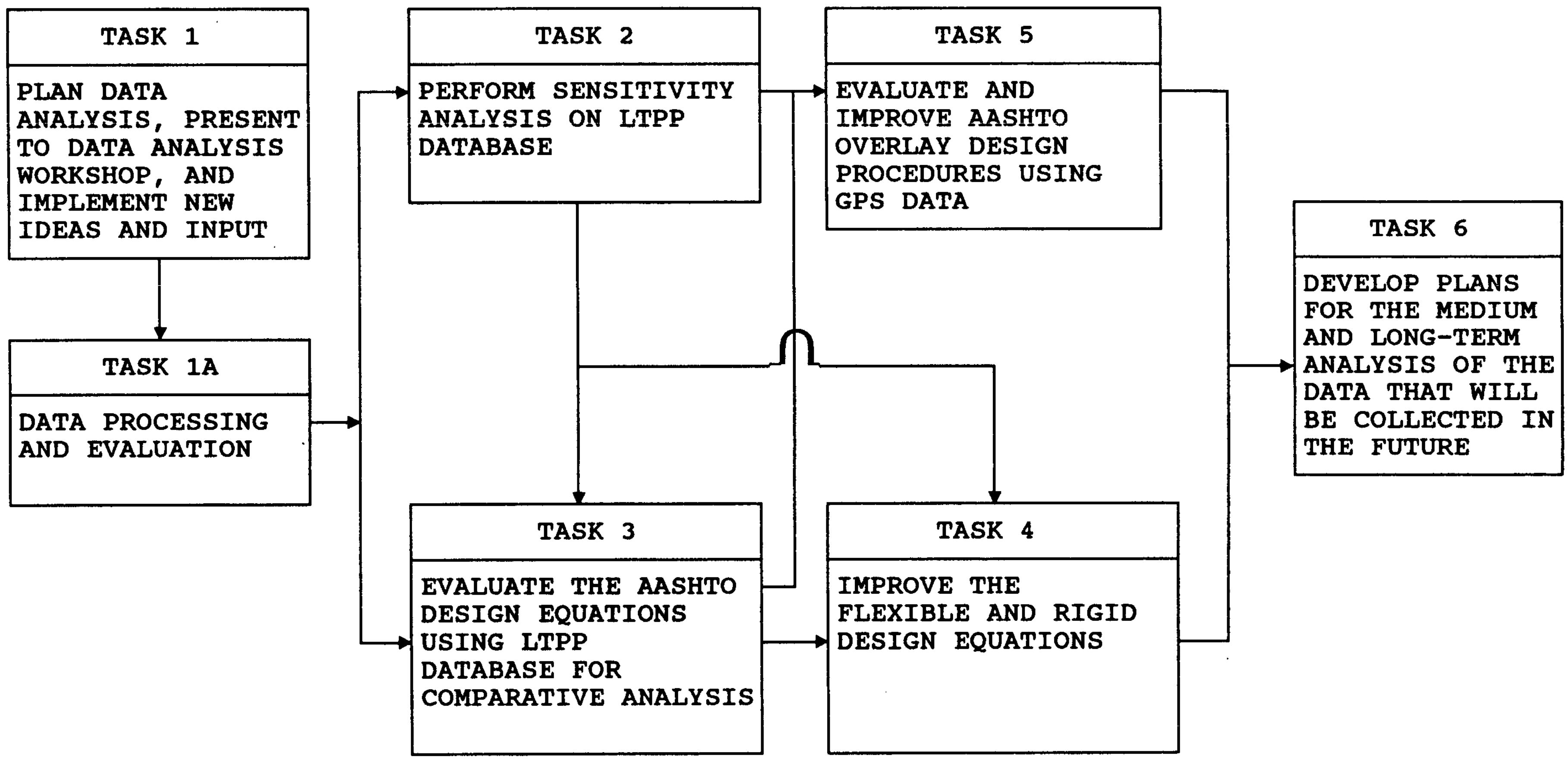

Figure 1.1 General Task Flow Diagram

It should be noted that some statisticians prefer to call the GPS experimental factorials "sampling templates" rather than experimental factorials, because existing in-service pavements were used instead of test sections that were constructed to satisfy rigorous experiment designs. Instead, the factorials were established to encourage reasonable distributions of parameters expected to be significant and test sections were sought to meet the factorial requirements. The Specific Pavement Studies (SPS) will in fact follow the requirements of designed experiments.

Table 1.1 Listing of SHRP LTPP General Pavement Studies (GPS) Experiments

GPS Experiment Number	Brief Description	No. of Projects in the Data Base
1	Asphalt Concrete Pavement with Granular Base	253
2	Asphalt Concrete Pavement with Bound Base	133
3	Jointed Plain Concrete Pavement (JPCP)	126
4	Jointed Reinforced Concrete Pavement (JRCP)	71
5	Continuously Reinforced Concrete Pavement (CRCP)	85
6A	AC Overlay of AC Pavement (Prior Condition Unknown)	61
6B	AC Overlay of AC Pavement (Prior Condition Known)	31
7A	AC Overlay of Concrete Pavement (Prior Condition Unknown)	34
7B	AC Overlay of Concrete Pavement (Prior Condition Known)	15
9	Unbonded PCC Overlays of Concrete Pavement	28

The environmental factors considered in the sampling templates were "freeze", "no-freeze", "wet", and "dry". These broad factors were applied to encourage selection of test sections with distributions of environmental variables. The four environmental zones (or regions) considered for selection of test sections appear in Figure 1.2. Where feasible, data sets for the individual distress types were further divided into four separate data bases by environmental regions and separate analyses conducted on each.

Analytical Products from Early Analyses

In order to plan future analyses, it is very useful to understand what products have been produced from these early analyses, under what the research staff views as very difficult circumstances. These are listed below:

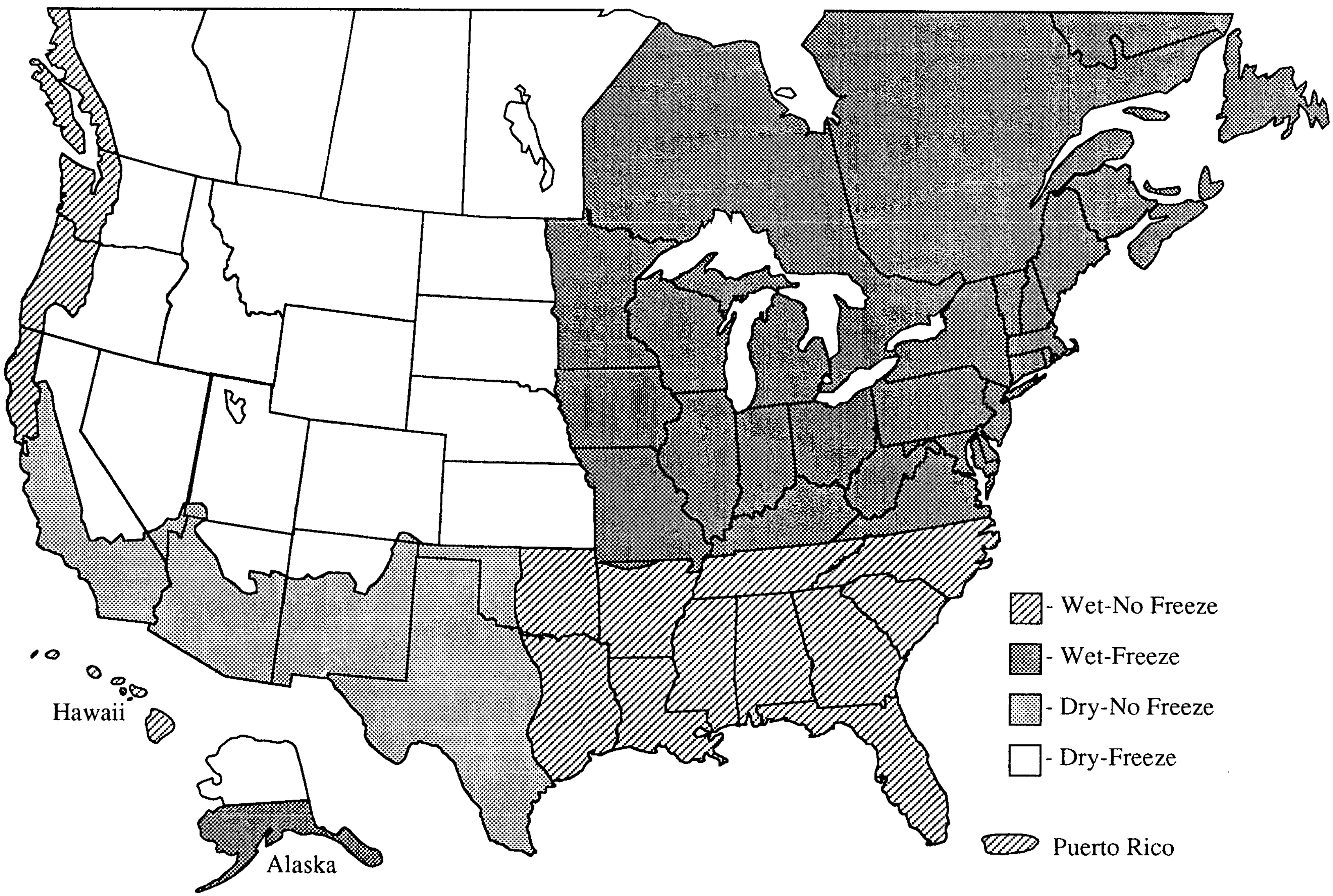

Figure 1.2. Environmental Zones for SHRP-LTPP Studies.

1. Data processing and evaluation:

   a. Useable data bases for combinations of pavement types and pavement distresses.

   b. Variable distributions

   c. Identification of test sections for which critical data is missing; i.e., those which cannot be used for analyses.

   d. Identification of biases.

   e. Interim distress models:

      a) regression models for all distresses adequately represented in the data base,

      b) valuable insight as to the validity of national models versus regional models,

      c) procedures for developing models from LTPP data.

2. Sensitivity analyses:

   a. Identification of significant independent variables affecting each distress model.

   b. Relative strengths of variables affecting each distress model.

   c. Procedures for conducting sensitivity analyses on LTPP data.

   d. Identification of procedures that do not work.

3. Guidelines for future applications:

   a. Identification of mechanistic variables and clusters for future modeling.

   b. Statistical methodology for LTPP data analysis.

4. Evaluation of AASHTO design equations:

   a. Evaluations of predictive capabilities of AASHTO design equations.

   b. Identification of shortcomings in AASHTO design equations.

   c. Identification of potential improvements to AASHTO design procedures.

5. Improvements to AASHTO Design Equations:

   a. Framework for using distress models directly instead of through the composite index "PSI".

   b. Preliminary distress models for interim use and as placeholders for future improved models.

6. Recommendations for future analyses (the subject of this volume).

Documenting Experience from Early Analyses

These early analyses were considered by many as primarily a means to sort out the data base in terms of its limitations and potential uses, and as pilot studies to aid in selection of analytical techniques for future analyses when more time sequence data are available. The objectives of the studies reported in this volume are to document the experience gained from the early analyses and to offer suggestions on the basis of this experience to future analysts. Volume 2 of this report will provide statistical information that will allow future analysts to evaluate the data that were available for these analyses. Much of what will be discussed appears in Volumes 3 and 4, but the intent is to pull all of this together in this volume. The results from all tasks to be discussed include:

1. data limitations and recommended actions to improve the data base and/or deal with limitations,

2. data expectations for future analyses,

3. identification of procedures that did not work and recommended procedures for the future,

4. discussion of potential modeling and analysis techniques for future use,

2

Data Analysis Procedures Used for Early Analysis (1992 and 1993)

Brief summaries of the procedures utilized for conducting both the sensitivity analyses and the evaluations of the AASHTO Design Guide equations are provided, along with discussions on the limitations associated with these analyses. Details are provided in Volumes 3 and 4. This information will aid future analysts of this data in selecting analytical procedures, identifying potential pitfalls to avoid, and recognizing limitations of the data.

Procedures for Sensitivity Analyses

"Sensitivity Analysis" does not have an established meaning for either research engineers or statisticians, but it has come to have a specific meaning to some individuals from both disciplines. The definition as applied to this research follows:

> "Sensitivity Analyses are statistical studies to determine the sensitivity of a dependent variable to variations in independent variables (sometimes called explanatory variables) over reasonable ranges".

There is no single method of conducting sensitivity analyses, but they all require development of reasonably adequate equations (or models) for predicting distress. In order to conduct successful sensitivity analyses of the type considered here, the equations to predict the distresses of interest must be statistically linear and must contain a minimum of collinearity between the independent variables. Predictive equations linear in the coefficients are required for sensitivity analyses because:

- Otherwise, the magnitudes of the effects from varying the individual independent variables would not be directly comparable,

- Nonlinear regression techniques are deficient in diagnostics to identify collinearity and influential observations. As collinearity must be minimized if the relative sensitivities are to be meaningful, use of nonlinear regressions could seriously limit confidence in the results,

- The research staff (including Dr. Pendleton, the statistical consultant) are not aware of any existing procedures for conducting sensitivity analyses on nonlinear models; therefore, it would have been necessary to develop a complex computer program which would have been far out of the scope and funding for these studies.

Procedures Used for Developing Distress Prediction Equations

A flow chart for the general procedures used to develop prediction equations appears in Figure 2.1. The selections of independent variables to be included in the studies are described in Chapter 2 of Volume 3. The selections of transformations of the variable (e.g., in logarithmic form rather than arithmetic) and interactions were primarily carried out as part of the multiple regressions themselves, which were part of the "multivariate analyses" indicated on Figure 2.1. All of the predictive equations for sensitivity analyses for HMAC pavements were developed using Statistical Analysis Systems (SAS™) software and those for PCC pavements were developed using the S-Plus™ software.

The univariate analyses involve studies to examine the data for potential distributional problems and anomalies. The purposes were to examine marginal distributions, identify gaps in the data, identify any unusual observations, and to identify functional forms. The procedures included:

1. Studies of continuous data descriptive statistics and frequency distributions.

2. Partitioning continuous variables by categorical ones.

The bivariate analyses were used to study pairs of data elements, which may turn out to be "unusual" although each variable itself is not "unusual". The specific expectations from the bivariate analyses were to identify two-variable relationships, bivariate unusual points, bivariate collinearities, data gaps, functional forms, and data "clusters".

The final step in the development of the predictive equations is termed, collectively, multivariate analyses. These analyses included studies to identify multivariate collinearities and the development of the pavement distress models. The procedures planned included:

1. Discriminant analysis to identify distressed from non-distressed pavements.

2. Development of regression analysis models for distressed pavements.

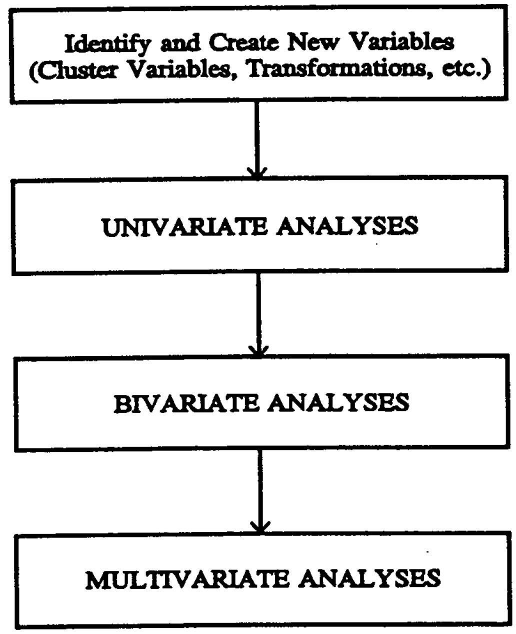

Figure 2.1 Flow Chart for Data Studies and Development of Equations to Predict Significant Distresses

3. Analysis of variance, comparing means of independent variables for distressed and non-distressed pavements.

The procedures described above were carried out as indicated, but it became apparent during the analyses that revisions and additions would be required. These are discussed in the next section of this chapter.

Principal component analysis was used to detect collinearity and influential observations. This method utilizes plots of eigenvector pairs to identify collinearities that may be masked by outliers, (for details, see Reference 1).

Products from these data evaluations that were used in the development of predictive equations included the following:

- Two-variable scatter plots
- Variable frequency distributions for the entire data base and by climatic zones
- Correlation matrices for the separate data sets individually
- Complete eigenanalysis for each data set individually
- Residual plots for trial equations
- Plots of predicted vs. actual distresses

It should be noted that the measured distresses for the inservice pavements do not include the period just after the lane is opened to traffic. Consequently, the early compaction of HMAC pavements from traffic is not directly represented. As a boundary condition, log KESALs was included in each equation as a separate term to enforce zero rutting with zero ESALs. Mathematically, the equations are undefined at zero ESALs; however, for practical purposes it is assumed that zero to some power is zero. The same boundary condition was enforced to insure zero change in IRI with zero ESALs, and age was used to enforce zero transverse cracking at the time of construction. Consequently, the predicted progression of distresses very early in an HMAC pavement's life is not reliable.

Problems Encountered and Modifications to Procedures

The procedures discussed above were first tested on rutting of HMAC pavements over granular base. As problems were encountered, this data set was used as a "test bed" for identifying problems and working out solutions, before continuing with data sets for other distress and pavement types.

As required for the sensitivity analyses, modeling was conducted using the least squares linear regression techniques which minimize the random error. This technique also assumes that the dependent variable is normally distributed about the regression line and that the independent variables are fixed and without error. It is believed that the distresses have approximately log-normal distributions about the regression line, and therefore the regressions were conducted to predict the common logarithm of the distress.

The first step was to analyze the individual independent variables using the SAS® all possible subset selection procedure. This procedure allows the user to offer a list of independent variables and the system will select which of these variables, singly and in combination, best predict the dependent variable. This procedure was not expected to give the final model; however, it was expected to aid in the determination of what variables were the most influential to prediction of the dependent variable.

The second step was similar to the first, except that all possible two and three-way interactions were tried in the regressions. The interactive terms were selected through consideration of the theoretical variable clusters discussed in Chapter 4 of Volume 3, terms appearing in prior distress equations, and engineering experience and judgement. When the sensitivity analyses were conducted on the resulting model, it became apparent that each independent variable needed to be either in log form or non-log form, but not in both.

The third step was to raise variables within the model to some power. The power was determined through an iterative process that found the models with the best root mean square error (RMSE), coefficient of determination (R^2), and "p-value" on the individual variable. The p-value is used to determine whether the independent variable is significant to the prediction of the dependent variable. However, the sensitivity analysis on the resulting equation did not produce logical or believable results. This led to serious discussions and experimentation, which then led to the conclusion that three-way interactions (containing three independent variables in a single term) and the powers of the variables were confounding the sensitivity analyses. It was decided to limit the models to main effects (single independent variables) and two-way interactions. While the fit of the resultant models was slightly (though not significantly) worse, the sensitivity results appeared much more logical.

To determine the stability of the model, regression analyses were completed on five different sets of 80% of the complete data set (a different 20% deleted each time), using the same equation form. The coefficients for each independent variable for each run were compared and found to be quite variable. If the equation had been stable, the coefficients would have been very similar.

Correlation among independent variables can lead to estimates of model coefficients that are illogical in sign. For example, when the variables "average monthly maximum temperature" and "annual number of days greater than 90F" were used in the equation for rutting, they had opposite signs. Although these non-intuitive model estimates do not generally mar the model's predictive ability, they are somewhat disconcerting to the practitioner and difficult to explain. At this point it was decided to try the technique of ridge regression (2), a statistical method which adjusts for collinearity (correlated independent variables) and produces more stable and logical model estimates. The parameter estimates in many cases change dramatically when using the ridge regression procedure. At some point during the iterative model developments the change in the parameter estimates becomes much smaller and this is the final equation that is used.

The terms in the equations previously checked by using five different sets of 80% of the data set had been primarily interactions between the independent variables. Thus, some of the main effects (individual independent variables) were added to the equation form and the regressions repeated, using a different set of 80% of the data set for each model. The comparisons of the five sets of coefficients proved to be much more consistent, indicating that the revised equation form fit the data better.

The model was found to contain certain highly correlated variables such as age and KESALs and subgrade moisture and annual precipitation. These pairs were identified and the variables in each pair with relatively low sensitivities were replaced by the variables with which they were correlated. The sensitivity analysis results for this equation were much more reasonable. The equation was then changed such that the other "half" of the "pair" was used in the interactions. The model created by using the variables with higher sensitivities produced much more reasonable results.

All of the models produced to this point and their resulting statistics were established from a data set that involved every environmental region (entire data set). In an effort to improve model statistics, the data were separated according to the four environmental zones used in the sampling templates and each data set was regressed using the equation form containing the better half of the pair. The results from some of the sensitivity analyses were not always reasonable. To try to remedy the problems encountered in the sensitivity analyses, models were (as described above) found using main effects alone. The R^2s decreased and RMSEs increased somewhat, however the results from the sensitivity analyses were more reasonable. Next, interactions that had previously been found to work well (included some of the three-way interactions) were added to the equations with just main effects. Values of R^2 and RMSE were improved but the results from the sensitivity analyses were not all reasonable. The above interactions were dropped from the equations and only two-way interactions were added. The values of R^2 and RMSE were not as good as those that included the three-way interactions, but were better than the equations with just main effects. The sensitivity analyses were more reasonable but still problematic, particularly for the wet-no freeze and dry-no freeze zones.

Coordination with the statistical consultant indicated that sufficient collinearity had not been expelled from the equation, so eigen analysis (see Appendix D of Volume 3) was used to identify additional variables to delete from the models. The ridge regressions mentioned previously were then used to develop new models. In each of the models, KESALs and structural thicknesses were "forced" into the equations. That is, the variables were placed in the models whether they improved the statistics or not.

It was concluded, at this point, that the five models produced, one for the entire data set and one each for each of the four environmental zones, were as good as could reasonably be expected within the constraints imposed by the requirements for sensitivity analyses and by limitations in the data sets available (primarily lack of adequate time sequence data for these early analyses). The techniques finally used for the rutting model were then adopted for other HMAC pavement distresses.

The procedure, arrived at by the experimentation described above for developing distress models to be used for sensitivity analyses, is further described in the next subsection. The modeling and sensitivity analyses are best combined as one process, so judgement can be applied to iterate toward the optimum models for use. The analyst must carefully reach a balance between: 1) expectations and knowledge from past research and 2) maintaining opportunity for the data to communicate new knowledge.

It should be noted that this procedure does not offer the best models that can be developed to predict pavement distress. It is likely that nonlinear regression techniques would result in better models, but these models would not have been practical for the sensitivity analyses, as sensitivity analyses for nonlinear models are much more complex and there are not any computer programs (like SAS® for linear models) to use in conducting them. However, this does not preclude common transformations, such as using logarithms or powers of the independent variables, as long as the equations are linear in the coefficients.

The procedures used for developing the PCC pavement models were essentially those discussed above. While the studies were in progress, the graphical capabilities of the S-Plus statistical software (3) was utilized to easily view scatter plots and three-dimensional plots of the data, which indicated relationships between all dependent and independent variables being considered. From observations of the previous two- and three-dimensional plots the explanatory variables that were not linearly related to the dependent variables were noted. Such variables were linearized by determining the best exponents for these variables. This was done using the Alternating Conditional

Expectations (ACE) algorithm introduced by Breiman and Friedman along with the Box-Cox transformation. Detailed descriptions of these techniques are provided in Reference 13.

These procedures were used to develop the final models that were used in the sensitivity analyses. In several cases, this general procedure had to be modified to meet the specific demands for the model to be developed as described in Volume 3.

Procedures for Establishing Sensitivities of Distresses to Variations in Significant Independent Variables

A variety of trials were run on the data set for rutting in HMAC on granular base. These procedures led to an algorithm that was consistently applied to determine all of the models. The algorithm appears in Figure 2.2 for HMAC pavements as a flow chart and is also described below:

1. Starting with log traffic, each single variable of the set of variables considered and its transformations are tried in the model. If a variable is found to improve the R^2, adjusted R^2, and RMSE without adding collinearity, it is allowed to stay. Once all the individual independent variables have been tried once, any that are not in the model at that point are tried again. For consistency's sake the first set of variables tried after log traffic are those dealing with the HMAC layers. Next, the variables identifying the base layers are tried, followed by the subgrade, and finally the environmental variables.

2. Once an equation with the main effects (variables identified as significant) has been established, other equations are tried that include two-way interactions of the main effects. When a trial interaction improves the R^2, the adjusted R^2, and the RMSE, but does not add collinearity, it is allowed to stay in the model. This is continued iteratively until all possible two-way interactions have been tried. Although techniques previously described were used to identify outliers, the analyst should be alert for other outliers that may be revealed as the analysis continues. It should be noted that main effects in some cases were replaced by interactions.

3. The ridge regression technique is then applied to stabilize the model, using the main effects and interactions that survived Step 2.

4. Conduct the sensitivity analysis on the final model as discussed above.

It should be understood that the calculated "sensitivities" that are assigned for the individual independent variables are very much dependent on the predictive equation itself. The values will vary, depending on the form of the equation and the set of independent variables included. As can be seen in Chapters 7 of Volume 3, models for different environmental zones can vary considerably in form and in variables that are significant to the prediction of a distress. Therefore, the relative sensitivities of the

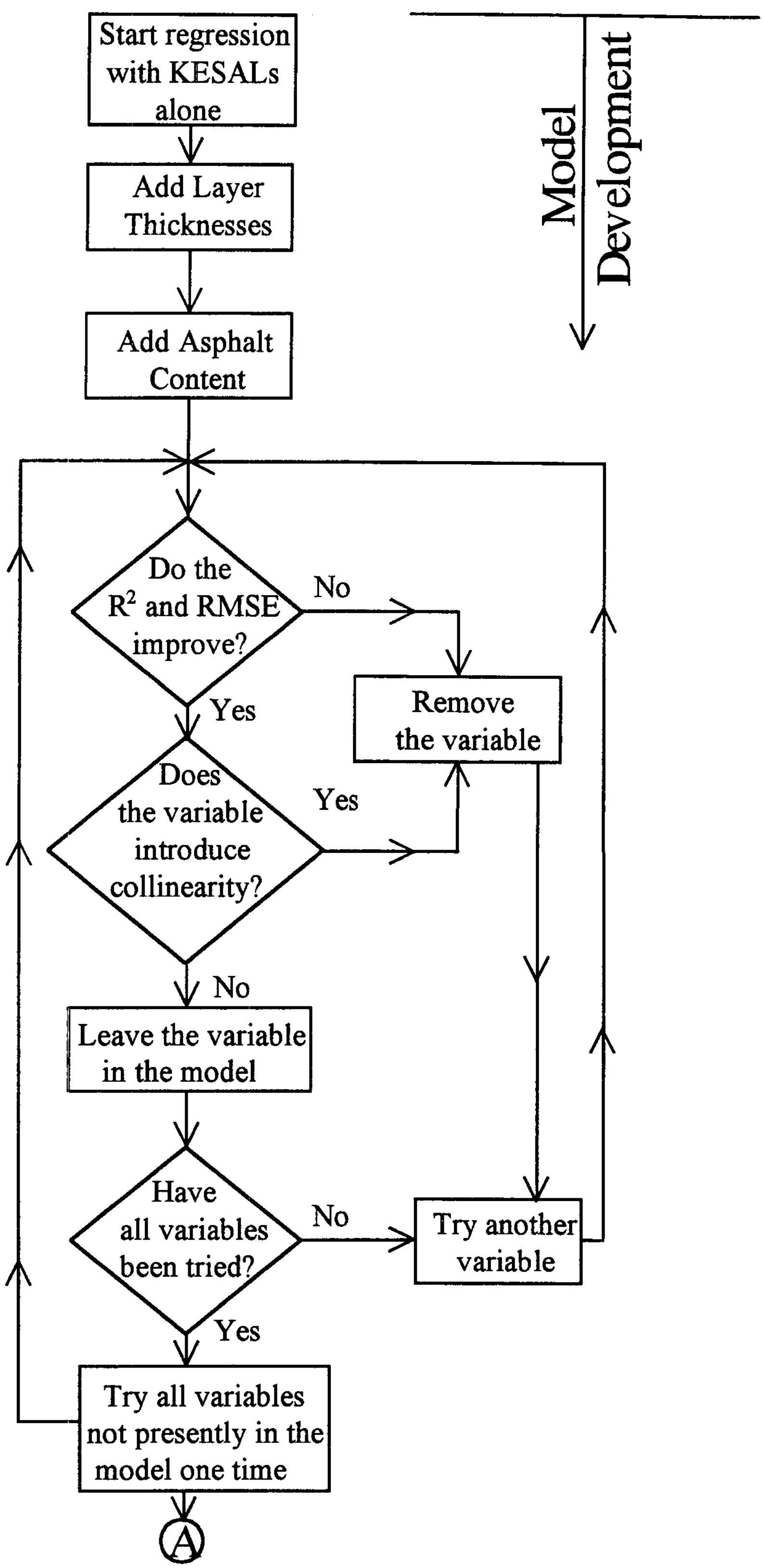

Figure 2.2 Flow Chart for Developing Distress Models and Conducting Sensitivity Analyses for HMAC Pavements

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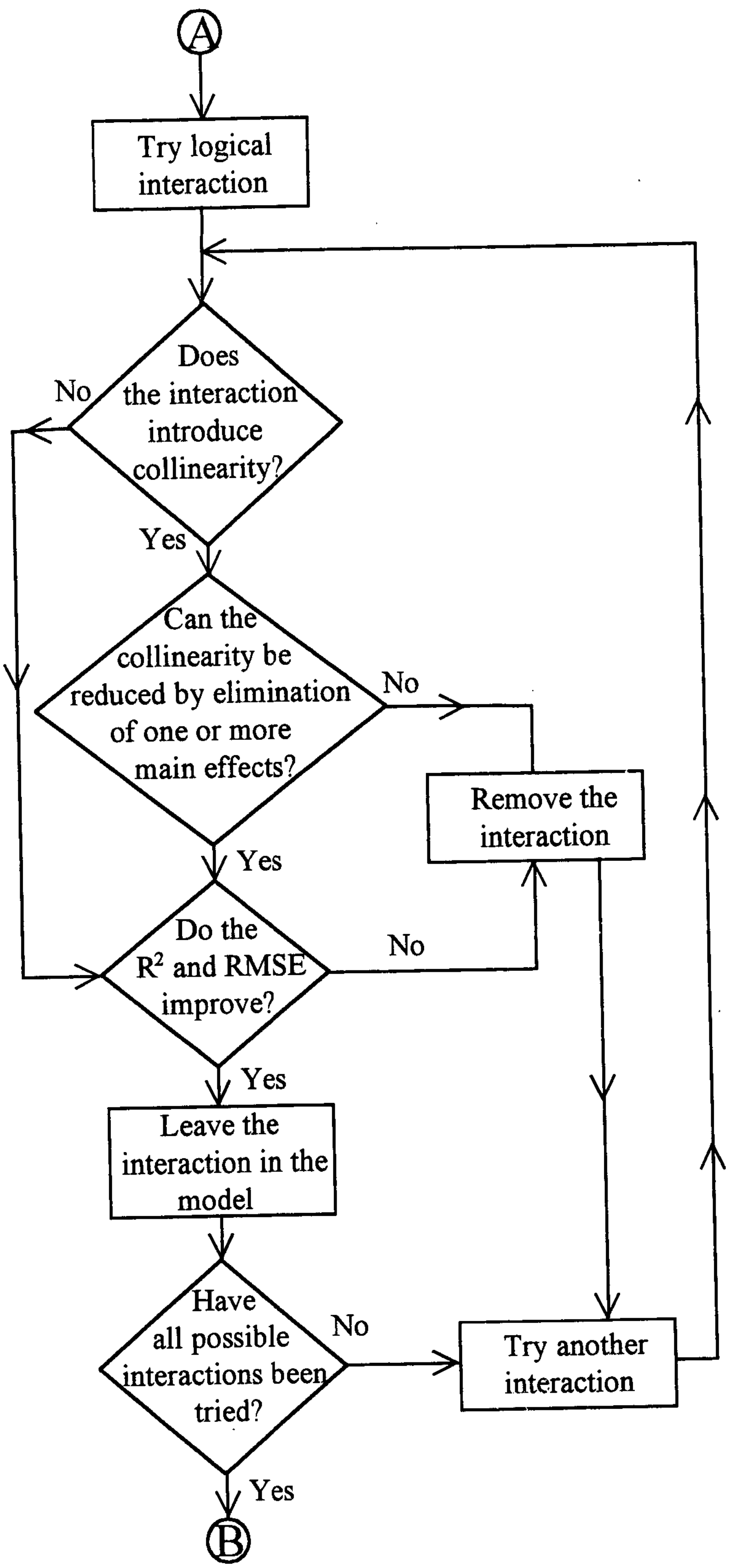

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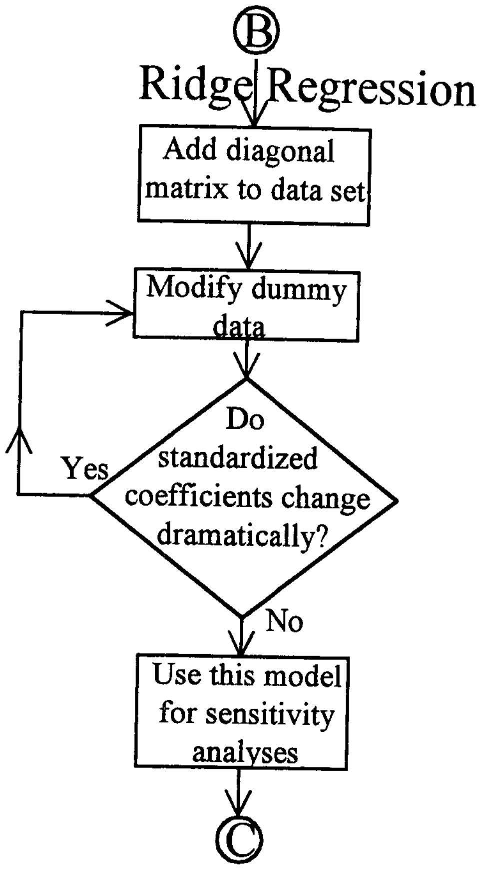

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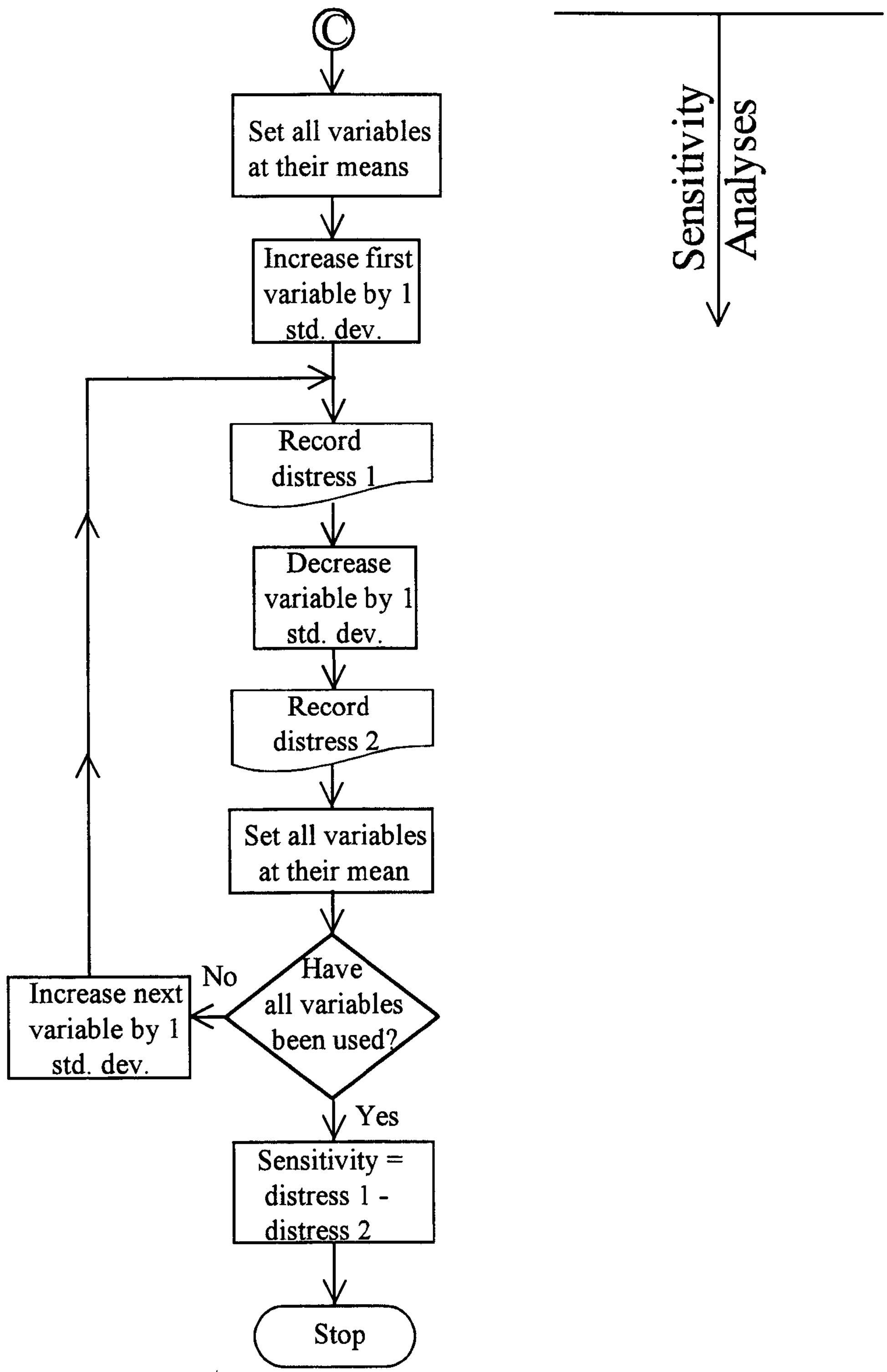
C
Set all variables at their means
Increase first variable by 1 std. dev.
Record distress 1
Decrease variable by 1 std. dev.
Record distress 2
Set all variables at their mean
Have all variables been used?
No
Increase next variable by 1 std. dev.
Yes
Sensitivity = distress 1 - distress 2
Stop
Sensitivity Analyses

independent variables should be considered indicative of their relative significance, rather than an absolute measure of the relative importance of the variables in terms of magnitudes. As obvious examples, it can be concluded that traffic, HMAC thickness, and rainfall merit consideration in design and pavement management, but their relative significances may vary somewhat for future equations.

While future predictive equations may be expected to be more precise and consequently offer higher confidence in the relative importance of individual variables, it is likely that truly precise evaluations may never be reached. It should suffice if the significant variables continue to be found significant in future analyses and are found to have more or less the same relative importance with relation to each other.

The only differences between the procedures used for the sensitivity analyses for HMAC and PCC pavements were in the modeling process, as discussed in the previous subsection and shown in Figure 2.3. The use of the S-Plus™ plotting capabilities and their "linearization" of the independent variables replaced the use of ridge regression and some of the iterations in the HMAC procedures discussed above. Once modeling had been completed, the same procedures were used to determine the sensitivities of the dependent variable to variations in the independent variables as were used for the HMAC data.

Table 2.1 Coefficients for Regression Equations Developed to Predict Rutting in HMAC on Granular Base for the Wet-Freeze Data set.

Rut Depth (Inches) $= N^B \, 10^C$ Where: N = Number of Cumulative KESALs

$$B = b_1 + b_2 x_1 + b_3 x_2 + \dots + b_n x_{n-1}$$
$$C = c_1 + c_2 x_1 + c_3 x_2 + \dots + c_n x_{n-1}$$

Explanatory Variable or Interaction (x_i)	Units	Coefficients for Terms In	
		b_i	c_i
Constant Term	--	0.183	0.0289
Log (Air Voids in HMAC)	% by Volume	0	-0.189
Log (HMAC Thickness)	Inches	0	-0.181
Log (HMAC Aggregate #4 Sieve)	% by Weight	0	-0.592
Asphalt Viscosity at 140°F	Poise	0	1.80×10^{-5}
Log (Base Thickness)	Inches	0	-0.0436
(Annual Precipitation *	Inches	0	
Freeze Index)	Degree-Days	0	3.23×10^{-6}

$R^2 = 0.73$ Adjusted $R^2 = 0.68$ RMSE = 0.19

As an example, the model developed for prediction of rutting in the wet-freeze environmental zone appears as Table 2.1. The form of the equation appears at the top of the table, with the explanatory variables or interactions appearing in the table, along with the coefficients that provide the details of the equation. As can be seen, the exponents B and C are calculated by multiplying the explanatory variables or interactions in the left column by the regression coefficients b_i and c_i and adding the results. For

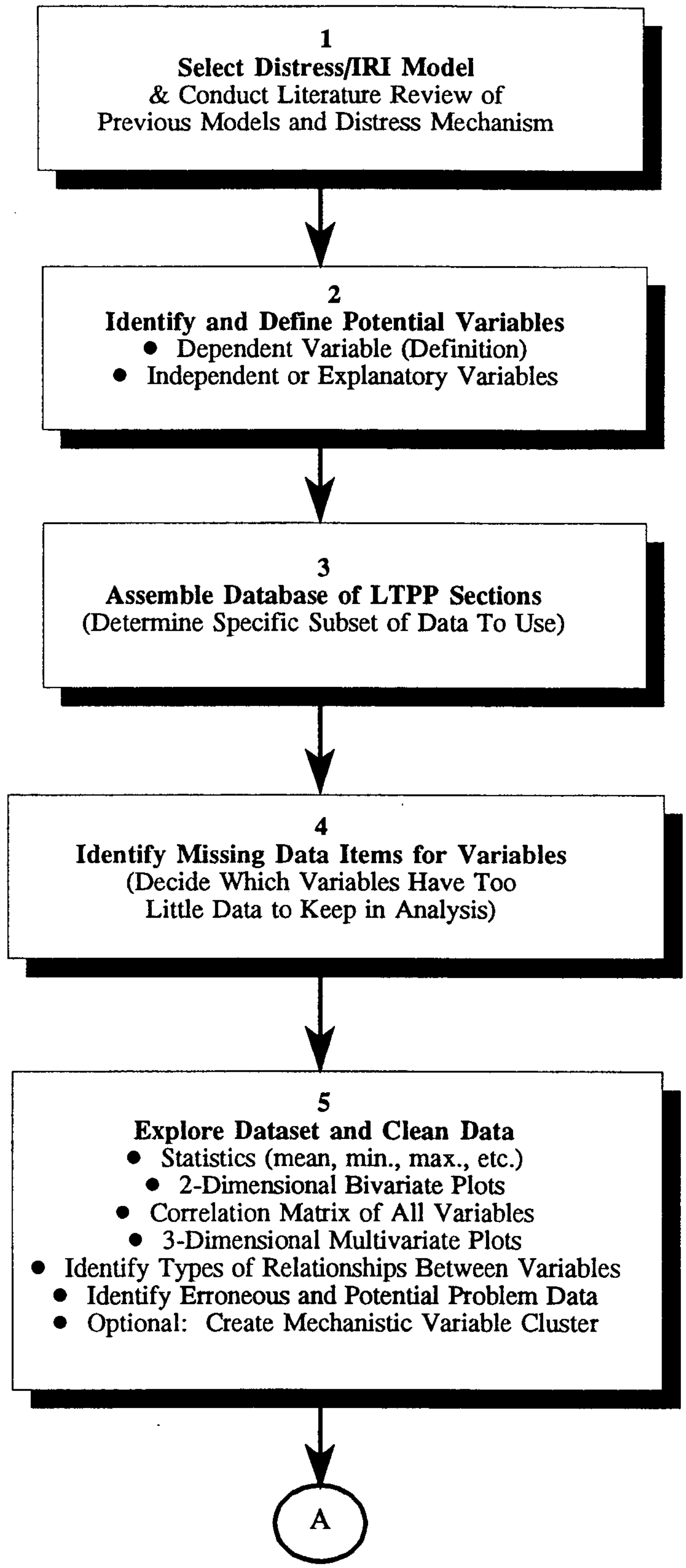

Figure 2.3 Flow Chart for Developing Distress/IRI Models for Concrete Pavements and Conducting Sensitivity Analysis

Continued on Page 28

A

6

Model Building

- Identify Functional Form of Distress/IRI with Traffic and Age
- Identify Boundary Conditions
- Select Potential Variables and Transformations for Initial Evaluation
- Conduct Linear Regression With All Variables (Observe Significance Levels, Collinearity, Residual and Predicted vs. Actual Plots)
- Add and Test Various Interactions
- Observe 2-D and 3-D Plots of Variables (Observe Relationships, Consider Additional Transformations)
- Identify Potential Outliers
- Conduct Further Regression Analyses
- Select Tentative Model

7

Sensitivity Analysis

- Set Variables at Their Means
- Increase/Decrease Each Variable by One Standard Deviation and Compute Distress/IRI for Each
- Sensitivity is Difference Between Plus and Minus Standard Deviation
- Repeat for All Variables
- Plot Sensitivity Graph
- Evaluate Reasonableness of Direction of Variables on Distress/IRI
- Evaluate Reasonableness of Sensitivity of Each Exploratory Variable
- Judge Adequacy of Tentative Model
- Revise Model if Deficient

example, the constant b_1 for this model is 0.183, and is equal to B because all of the other values of b_i are 0. To calculate C, the constant term is 0.0289, the log of air voids in HMAC is multiplied by -0.189, etc. The results of the sensitivity analyses conducted with this predictive equation appear as Figure 2.4a. From Figure 2.4a, it can be seen that the strongest impact on the occurrence of rutting in the wet-freeze environment may be expected to be the number of KESALs. The dashed lines to the left indicate that reductions in KESALs decrease rutting, but the standard deviation for KESALs is greater than the mean and negative KESALs are not possible. Freeze index is the next most important, followed by percent of the HMAC aggregate passing a #4 sieve, air voids, etc. It can also be seen from the directions of the arrows that increasing KESALs and freeze index may be expected to increase rut depths, while increasing amounts of aggregate passing the #4 sieve, air voids, and asphalt thickness may be expected to decrease rutting.

In order to illustrate how different the sensitivities may be from one environmental region to another, the sensitivity analysis results for the dry-no freeze environmental zone are included as Figure 2.4b. As can be seen, the majority of the variables are the same as for the wet-freeze environment, but there are some differences and the relative levels of sensitivities vary between environmental zones.

While the sensitivity analyses offer very useful insight, it must be remembered that most of these pavements are in very good shape, so the interactive effects of water seeping through cracks and expediting deterioration in lower layers is really not represented here.

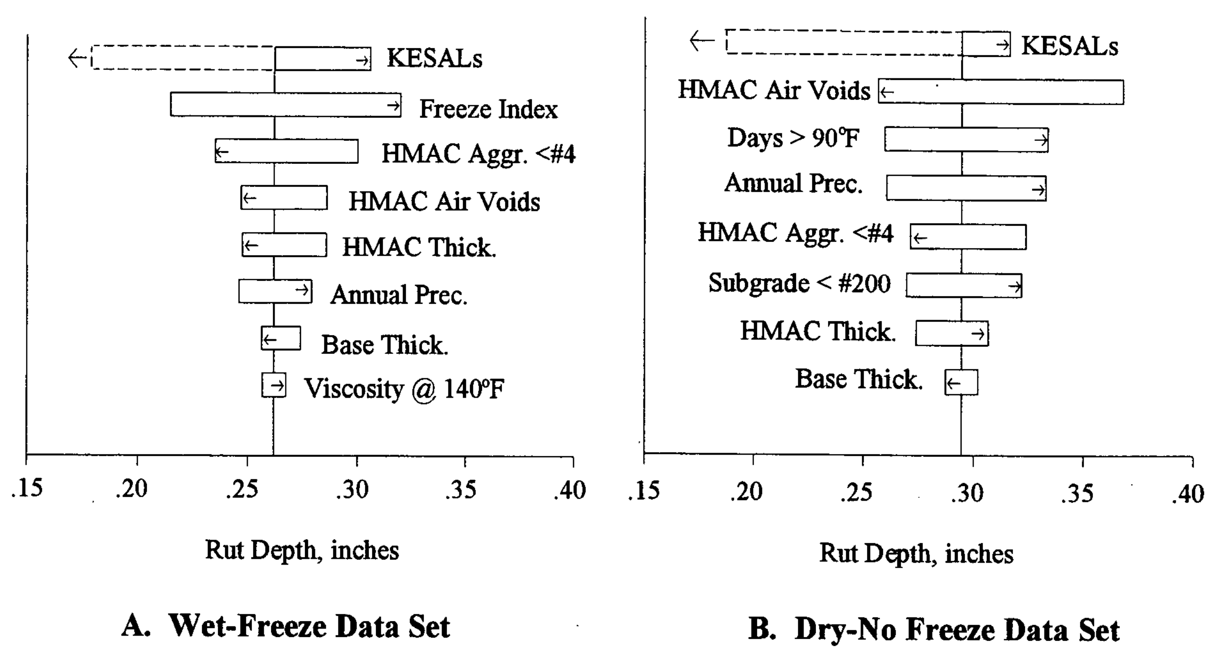

Figure 2.4 Results from Sensitivity Analysis for Rutting in HMAC on Granular Base

Limitations to Sensitivity Analyses Resulting From Data Shortcomings

This project involved the analysis of data observed on in-service pavements. The early results obtained depended completely on the adequacy of the data base from which they were developed. Therefore, it is important to discuss the data resources available to the research team, with some discussion of those that will be available to future analysts. There are certain limitations to the studies that are an unavoidable consequence of the timing of the early data analyses. For instance, much better traffic data will be available for future data analysts from the monitoring equipment recently installed. This early data analysis was based on estimates of past Equivalent Single Axle Loads (ESALs) of very limited accuracy. While years of time-sequence monitoring data will be available later, these early analyses have distress measurements for only one point in time, or at most two. For most distresses, an additional data point may be inferred for conditions just after construction; e.g., rutting, cracking, faulting of joints, etc., may generally be taken as zero initially. Analyses for roughness increases depended for most test sections on educated estimates or the use of averages for initial roughness (derived from SHA estimates of initial PSI).

The distribution in ages of the test sections offered some assistance in overcoming the lack of time sequence data. As an example, Figure 2.5 shows the distribution of pavement ages for the GPS-1 experiment, Asphalt Concrete over Granular Base. A number of test sections are represented in all time intervals through 20 years of age.

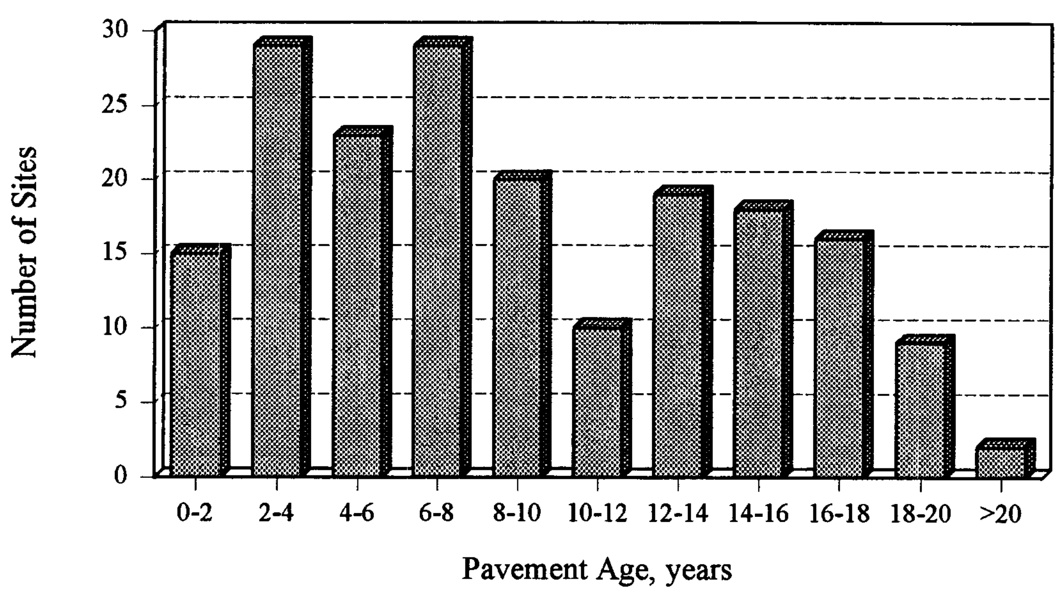

Figure 2.5 Distribution of Pavement Age, Experiment GPS-1, AC over Granular Base

Another shortcoming of the data bases that influenced the results were missing items of inventory data, that data from SHA project files that concern the design and construction of the pavements. Some data elements were available for all of the test sections, while others for some test sections were not known and could not be found. Unfortunately, it will generally not be possible to obtain this missing inventory data so they will be missing for future analyses as well.

The plans developed for these analyses were well accepted, but during the processing and evaluation of the data, it became apparent that all of the plans could not be carried out. Many of the test sections had not experienced distresses as yet, and those that had generally would have only 1 or 2 distress types. The only type of distress that was generally available for all test sections was roughness, and it was necessary to estimate the initial roughness in order to study increases in roughness. For HMAC pavements, rutting information was also available for nearly all test sections. It was not possible to study alligator cracking in flexible pavements, because there were only 18 test sections reported to have any alligator fatigue cracking. Similarly, ravelling and weathering could not be studied as only 3 test sections had experienced this distress. The only three distress types for flexible pavements for which sufficient data were available to support the studies were rutting, change in roughness (measured as International Roughness Index), and transverse (or thermal) cracking.

Predictive models for PCC pavements were developed for 10 combinations of pavement type and distress type. These models included joint faulting for doweled and non-doweled joints; transverse cracking for JPCP; transverse crack deterioration for JRCP; joint spalling for JPCP and for JRCP; and IRI for doweled JPCP, non-doweled JPCP, JRCP, and CRCP. However, sufficient data were not available to allow development of regional models, as the number of total test sections that could be used for a specific combination of pavement type and distress varied from only 21 to only 59. However, more test sections should be available for future analyses and data quality will be enhanced by time sequence observations.

Friction loss was also eliminated from the studies because there were only 3 data elements to use for independent variables in addition to ESALs and none of them would be expected to relate closely to the polishing of aggregates. Also, initial friction values were not available and would have to be estimated in order to study friction loss. If this is to be studied in future analyses, it may be necessary to obtain more data elements (such as polish values for aggregates).

The study of overlaid pavements was to have been of high priority, but it was generally agreed that pavement condition prior to overlay was a critically important variable and this was not available for pavements that were overlaid prior to entering the GPS studies. It was decided early in the implementation of the LTPP studies that test sections would be sought for pavements for which overlays were imminent, such that the condition prior to overlay would be available. A number of such test sections have been implemented, but none of those are old enough to have appreciable distress. The total numbers of overlaid pavements were very limited, and for the reasons discussed above only a few had sufficient information for successful analyses. Consequently, analyses for the overlaid pavements have been limited to the studies in Task 5; i.e., used only for evaluating the AASHTO Overlay Design Equations (see Volume 4). Hopefully, there will be more overlaid test sections and more distress for future studies.

It was also proposed that current knowledge be integrated into the analyses by use of mechanistic clusters of variables in the regression equations to predict distresses, which would then be used to conduct the sensitivity analyses. This plan to utilize mechanistic clusters of variables, based on theory, was thwarted for HMAC pavements by lack of layer stiffness data, which only started to become available in the fall of 1992, and still were not all available as this report was being written. As the mechanistic theory required layer moduli of elasticity, use of mechanistic clusters was limited to providing guidance for organizing interaction terms to try in the multiple regressions for developing predictive equations for distresses. Layer moduli will be available for future studies, so mechanistic clusters may be used as previously planned. For PCC pavements, the layer moduli were backcalculated from FWD deflections. The backcalculated foundation k-value was used to develop an illustrative mechanistic-empirical model for cracking of JPCP. A relationship between cumulative fatigue damage and transverse slab cracking was obtained. Fatigue damage over time was calculated from slab stress, strength, traffic loadings and other inputs.

As with any data analysis, the analysis staff had to be concerned about potential biases in the data bases. Several areas of concern identified by Mr. Paul Benson, a member of the Expert Task Group for Experiment Design and Analysis, were: 1) imbalances in the number of sections provided by different states, leading to possible undue influence from one state's design, construction, and maintenance practices; 2) the possibility of systematic differences in the interpretation of SHRP guidelines for test section selection by the states and by the four SHRP regional offices and their engineers; 3) uneven distribution of test sections in experimental factorials; 4) the possibility that by allowing older non-overlaid pavements we are selecting the "survivors", which are not typical of pavements in general; and 5) in a similar vein, the possibility that by basing much of our analysis on older pavements we may not be reflecting changes already made in modern construction and design practices. The following recommendations by Mr. Benson were followed in the analyses:

1. Limit the inference space of a model where the data are limited or grossly unbalanced. Consider regional models where the data do not warrant a national model.

2. Combine experiments (where distress mechanisms may be similar) to achieve better balance (specifically Experiments 1 and 2 and 3 and 4).

3. Examine the distributions of independent and dependent variables for non-normality, bi-modulism, and extreme values; where such are found, attempt to determine their source.

4. Conduct a thorough residual examination as soon as preliminary models are available, comparing residuals to project age, state, season tested, and others to determine possible sources of bias.

Other biases noted during evaluation of the data were:

1. air voids were generally lower for HMAC test sections in the dry-freeze region than in the other regions.

2. the subgrades for test sections in all data sets were mostly coarse-grained; e.g., the effects from clay subgrades were not adequately represented. (This bias can only be corrected by purposely recruiting additional test sections with clay subgrades).

3. only a few sections with severe PCC durability cracking were included. However, any of the new pavements could develop durability problems. Current specifications are aimed at eliminating PCC durability problems, so the models derived should be useful for design purposes.

The establishment of project selection criteria for the GPS in 1986 necessarily introduced biases that precluded some pavements in the overall U.S. and Canadian population of pavements. The intent was to create an inference space that represented the great majority of the overall population, and that would not be complicated by pavements with extreme or abnormal properties (severe D cracking, steep slopes, sharp curves, etc.). A "white paper", written by Dr. J. Brent Rauhut in approximately May 1986 as a basis for evaluating potential biases, is included as Appendix A. The evaluation of the potential biases did not result in any change in the project selection instructions. Also, it became apparent later that the State Highway Agencies had ignored some of the instructions in a small number of cases.

There has been no effort (for practical reasons) to coordinate the times when monitoring data were collected. Because of the time required to accomplish all the monitoring activities and the impracticability of monitoring in cold climates in the winter, times for monitoring have been essentially random. It is understood that responses to loads and environment vary seasonally, and that the manifestations of distress can also vary seasonally (e.g., cracks in HMAC pavements that are visible in cold weather may seem to "heal" in hot weather). Seasonable effects could not be studied in these analyses, but the FHWA is presently (1993) implementing seasonal monitoring of 64 GPS test sections with various pavement structures and varying environments. These test sections will be instrumented to monitor environmental data elements, and will be monitored monthly during alternate years (half one year and the other half the next, etc. for years to come). Results from these studies may allow consideration of seasonal effects in future analyses.

Procedures for Evaluation of and Improvements to the AASHTO Flexible Pavement Design Equations

Procedures Used for Evaluation

The equation evaluated was the one in the 1986 "AASHTO Guide for Design of Pavements" (4) below:

$$\text{Log } W = Z_R * S_o + \text{Log } \rho + \frac{G_t}{\beta} + 2.32 \text{ Log } M_r - 8.07 \qquad (2.1)$$

Where:

G_t = β (log W - logρ) = Log (Δ PSI/2.7),
W = the number of 18 kip ESALs,
ρ = $0.64\ (SN + 1)^{9.36}$,
β = $0.4 + 1094/(SN + 1)^{5.19}$,
SN = $a_1 D_1 + a_2 D_2 m_2 + A_3 D_3 m_3 + \ldots\ldots + a_n D_n m_n$,
D_i = thickness of layer i, in.,
a_i = structural coefficient for the material in Layer i,
m_i = drainage coefficient for the material in Layer i,
Z_R = standard normal deviate,
S_o = overall standard deviation,
M_r = resilient modulus (psi).

As this equation was used for research instead of design, a 50% reliability was assumed, which resulted in $Z_R = 0$.

The original equation for calculating current Pavement Serviceability Index (PSI) was reported in the AASHO Road Test Report 5 (5) as follows:

$$PSI = 5.03 - 1.91 \log (1 + \overline{sv}) - 1.38\overline{rd^2} - 0.01 \sqrt{c+p} \qquad (2.2)$$

where:

$\overline{sv}$ = the average slope variance as collected using the CHLOE profilograph,
$\overline{rd}$ = the average rut depth based on a 4-ft. straight edge,
c = the square feet of Class 2 and Class 3 cracking per 1,000 ft,^{2}
p = bituminous patching in square feet per 1,000 ft.^{2}

This equation, commonly used in the past for estimating PSI, was used to determine current PSI with values of slope variance derived from surface profiles measured with a GM profilometer and rut depths measured by PASCO's RoadRecon unit. Cracking and patching were not included in the calculation of the current PSI. Significant quantities of cracking and patching were noted on only a few of the test sections, and the impact of this term was not considered to be significant considering that its coefficient is only 0.01. The mean value of current PSI was 3.53, with a standard deviation of 0.49.

Observed PSI loss is then the difference between the initial PSI and the current PSI calculated using Equation 2.2 above. The mean value for observed PSI loss was 0.70 and the standard deviation was 0.51. Initial values of PSI were estimated by the State Highway Agencies, resulting in a mean value of 4.25 and a standard deviation of 0.23.

Equation 2.1 was used to predict the total KESALs required to cause the observed losses in PSI. Rearranging Equation 2.1 slightly results in:

$$\Delta\ PSI = 2.7 \left(\frac{W}{\rho S_m}\right)^{\beta} \tag{2.3}$$

where: $S_m = (M_r)^{2.32} * 10^{-8.07}$

Using Equation 2.3, the predicted PSI losses caused by the traffic estimated by the State Highway Agencies (SHAs) were calculated.

Resilient moduli for the subgrade (M_r) were represented by the stiffnesses backcalculated using the procedures in the 1986 Guide (4), which are based on use of deflections measured by an outer sensor of a falling weight deflectometer. Historical traffic data provided by the SHAs were used for the traffic data (W) in these calculations. The cumulative KESALs (1,000 ESALs) for each section were divided by the number of years since the test section was opened to traffic to obtain average values per year. This allowed extrapolation of the extra year or two beyond 1989 to estimate a traffic level associated with the dates of monitoring activities. Most of the monitoring data used were obtained in 1990 or 1991.

Figure 2.6 is a plot of KESALs estimated by Equation 2.3 to cause the observed losses in PSI versus SHA estimates of KESALs. As can be seen, the KESALs predicted by Equation 3 were consistently much higher than those estimated by the SHAs. Only 9 of the 244 predictions were lower than the SHA estimates, while the predictions were over 100 times the SHA estimates for 112 test sections. As the predictions from the design equation appeared to be very poor for in-service pavements, the thrust of the research turned toward identifying its problems and developing more reliable equations. The work activities comprising the evaluation consisted of a number of separate analyses, depicted in flowchart form in Figure 2.7 (details in Volume 4).

As partial explanation, it was noted that 74% of the in-service test sections in this study had experienced a loss in PSI of 1.0 or less, whereas those at the road test experienced losses of 2 to 3. Further, the average absolute deviation of observed PSI from the computed curves at the AASHO Road Test was 0.46 (6), so some 39% of the in-service test sections in this study have currently experienced losses of PSI within the "noise" at the road test.

Linear regressions were conducted on the data base, using the form of Equation 2.1. This resulted in an R^2 of 0.09, indicating that the equation form simply did not represent in-service pavement performance. Additional factorial studies indicated that the equation appears to falter for structural numbers less than 3, cumulative traffic greater than 5 million ESALs, or subgrade moduli greater than 10,000 psi (a value of 3,000 psi was used for the analyses of the road test data); that is, for conditions outside the inference space of the AASHO Road Test.

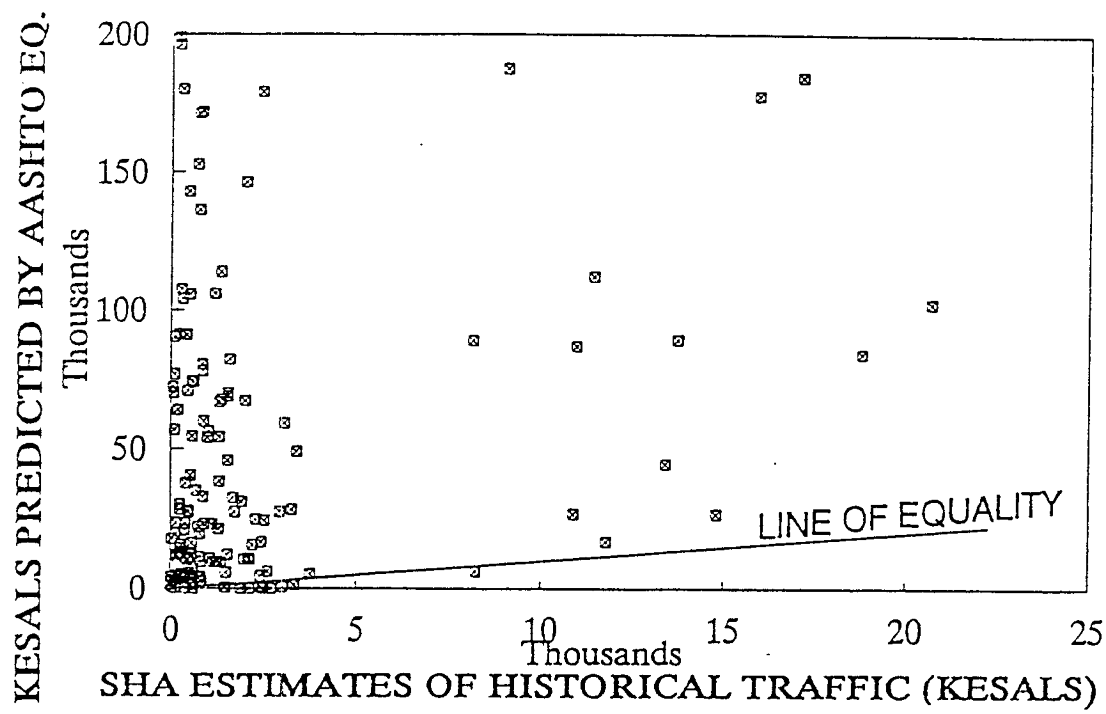

Figure 2.6. Comparisons of Predicted vs. "Observed" Traffic Loss to Cause Observed PSI Loss

Linear regressions were also conducted to model the ratio of predicted to observed traffic. This resulted in a model with R^2 of 0.77, which included structural number, subgrade modulus, and PSI loss (variables in Equation 3), but also included average annual rainfall and average number of days below freezing. Apparently unsuccessful attempts have been made through the years to extrapolate the equation outside its inference space, as there are obvious differences in environment and subgrade between Ottawa, Illinois and the rest of North America.

The backcalculated subgrade moduli appeared to be quite high, but laboratory testing for resilient moduli was just getting started when these analyses were being conducted. However, comparisons to 106 laboratory results late in the analyses indicated that the mean ratio of backcalculated to laboratory -derived moduli was 4.48, with a standard deviation of 2.47. These 106 laboratory moduli were substituted for the backcalculated moduli and the ratios of predicted to observed ESALs were considerably decreased.

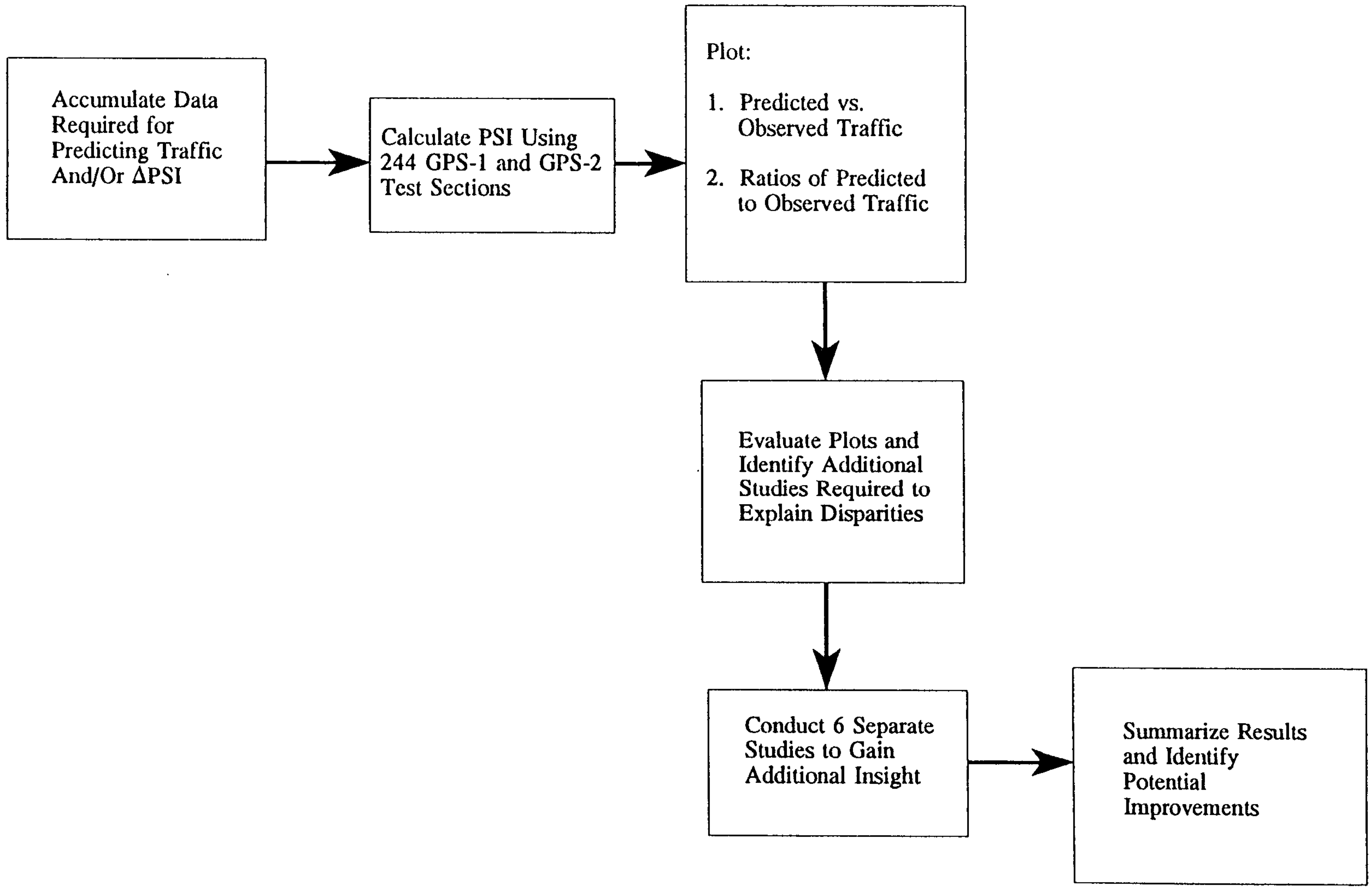

Figure 2.7. Work Plan for Evaluating the AASHTO Flexible Pavement Design

Figure 2.8 shows the ratios of predicted to observed KESALs with both backcalculated and laboratory-derived subgrade moduli. As can be seen, the number of "reasonable predictions" with ratios of 2 or less changed from 13 using the backcalculated subgrade moduli to 60 using the laboratory moduli. While the predictions improved greatly, the ratios for 46 predictions still ranged from 2 to over 100, corroborating the weaknesses in the equation noted throughout the studies. It appears certain that future design equations must take into account differences between backcalculated and laboratory-derived resilient moduli, as well as more accurate treatment of other significant variables.

It was concluded that the AASHTO design equation for flexible pavements is a poor prediction equation for inservice highway pavements, and that its use will very frequently result in unconservative designs.

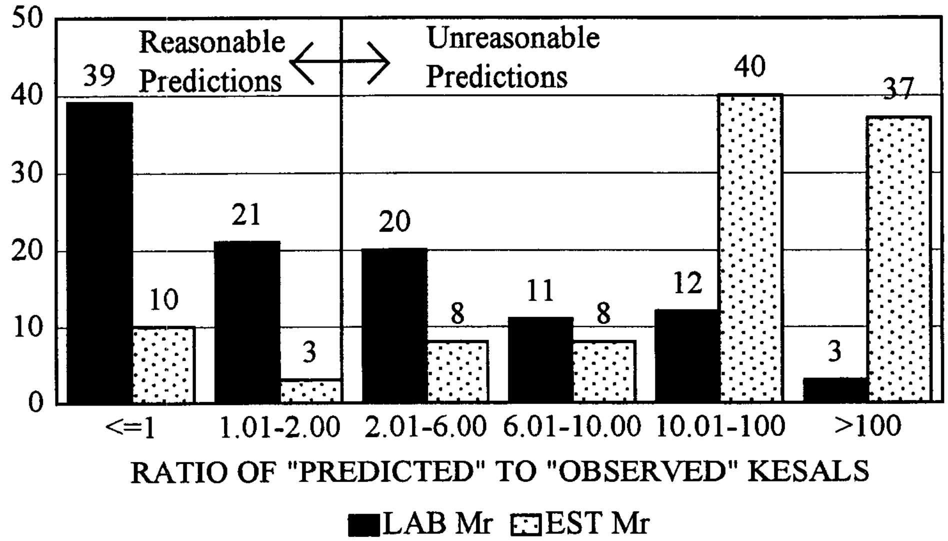

Figure 2.8. Numbers of Test Sections Having Different Ratios of Predicted to Observed ESALs, When Subgrade Moduli from Laboratory Testing or Backcalculations are Used in the Design Mix.

Procedures Used to Develop Improved HMAC Pavement Design Equations

It became apparent early in the research that many in the highway community were not interested in continuing use of the "composite index" called Present Serviceability Index for design. The preference was for separate design equations for the several significant distress types, so that they could be used both for pavement management and for balanced designs to minimize the distresses individually. The distress types considered to be significant are alligator cracking, rutting, transverse (or thermal) cracking, increases in roughness, and loss of surface friction. However, alligator cracking could not be studied at this early stage as their were only 18 pavements displaying that distress, and the data collected were not considered adequate for modeling loss of surface friction.

The original intent was to rearrange the models developed for the sensitivity analyses as design equations, but separate consideration of HMAC and unbound base thicknesses was problematical, as the separate effects for some distress types and environmental zones were not additive. That is, increasing thickness of either did not necessarily result in a decreased required thickness for the other. Consequently, it was decided to use structural number, in lieu of HMAC and unbound base thicknesses separately, to develop models that were better behaved.

The models were developed again with structural number, but the results discussed above were still reflected in the design models. These models for separate environmental zones had values of adjusted R^2 that varied from 0.69 to 0.88. These models appear in Volume 3, and are similar in format to the example in Table 2.1.

While these models may prove over time to be reasonable, they are based for this early analysis on very limited time sequence data (generally an initial point and another in 1990 or 1991 for the distresses) and should be used with care and only as design checks in concert with other design procedures. While a good distribution of pavement ages undoubtedly helped in explaining "curvature" in the relationship that will be enhanced by future time sequence data, the research staff does not wish to promote these models for general use at this time.

Limitations Imposed by HMAC Data Base Shortcomings

Most of the limitations noted above for the database relative to the sensitivity analyses were limitations here as well, except for lack of time sequence data. In addition to the limitations noted above, however, the lack of measured initial serviceability values, subgrade stiffnesses, and data relative to the subgrade volume changes limited these analyses.

Measured initial values of serviceability index (when the pavements were open to traffic) were not available. Few State Highway Agencies measured roughness of new pavements in the past. With this in mind, it was necessary to utilize either estimates from the SHAs or estimates arrived at by other means. Most values assigned were comparable to those used for the initial serviceability indices at the AASHO Road Test (4.5 for rigid pavements and 4.2 for flexible pavements). Unfortunately, the current estimates of initial PSI are not expected to improve for future studies.

The only complete set of subgrade stiffness data available was that backcalculated from deflection data for the outer sensors. This was calculated using procedures specified in the 1986 AASHTO Guide. Very little laboratory and backcalculated subgrade moduli (using all FWD sensors) were available for the studies. (Later comparative studies of backcalculated moduli to moduli measured in SHRP laboratories indicated very little correlation between moduli derived by the two methods). Both backcalculated moduli (using all 7 sensors) and laboratory resilient moduli will be available for use in future analyses.

Subgrade volume changes are also considered to be a significant contributor to changes in the serviceability index. Sufficient information to adequately address these potential losses, however, was not available. In particular, the depths of frost penetration for estimated serviceability loss due to frost heave was not included in any of the environmental data collected, nor could it be readily calculated or estimated based on the data available. Similarly, "soil fabric" data required for estimating the swell rate constant was not available in the data base. Crude estimates were made of these values on a subjective basis to complete the evaluations. As it turned out, there are few sections with a fine-grained subgrade soil that would be impacted by such subgrade soil volume changes. Should these evaluations be pursued further in the future, however, additional information to support the generation of these factors should be collected. Additional sections with fine subgrade, as well as the required data to evaluate the subgrade volume changes, will be needed if further evaluations of the subgrade volume changes are to be pursued.

It should be noted that the slope variance information utilized to calculate the serviceability indices is not identical to that collected from the CHLOE profilograph. Considering the distinctions between the slope variance generated as a summary statistic from the profile collected by the GM Profilometer and that of the slope variance generated at the road test, there is a potential for some error in calculated serviceability loss as a result of these distinctions. Although this discrepancy was not considered substantial, it is a potential limitation on the precision of these results.

While not specifically data limitations, the following two facts considerably limited the evaluations:

1. The accelerated trafficking to "failure" at the road test is not representative of in-service HMAC pavements. Pavement engineers typically intercede with overlays or other rehabilitation long before serviceability loss approaches the levels considered as failure at the road test.

2. The subgrade elastic moduli was assumed to be 3,000 psi for the development of equations for HMAC pavements at the road test, whereas very much higher moduli result from backcalculation or current laboratory protocols. Use of these moduli in the AASHTO design equation resulted in much greater predictions of ESALs than had actually occurred.

Procedures for Evaluation of and Improvements To The AASHTO Rigid Pavement Design Equation

The analyses were carried out using the original AASHTO design equation and the 1986 extension of the original design equation (unchanged in the 1993 guide). The analysis using the AASHTO original equation was mainly done to determine if the improvements to the prediction model were beneficial.

The AASHTO design equations were evaluated by comparing the predicted 18-Kip (80 kN) Equivalent Single Axle Loads (ESALs) for each test section determined from the design equation, to the "observed" ESALs (estimated from traffic data) carried by the section. The predicted ESALs are calculated using the concrete pavement equations from the original Road Test and the latest extended form in the 1986 AASHTO Design Guide for Pavement Structures (4).

The original 1960 AASHTO design equation is a relationship between serviceability loss, axle loads and types and slab thickness:

$$G_t = \beta(\log W_t - \log_{\rho}) = \log\left(\frac{4.5-p_t}{4.5-1.5}\right) \tag{2.4}$$

Where:

- G_t = the logarithm of the ratio of loss in serviceability at time t to the potential loss taken to a point where serviceability equals 1.5.
- β = a function of design and load variables that influence the shape of the p-versus-W serviceability curve.

=

- W_t = cumulative 18-kip ESALs applied at end of time t.
- ρ = a function of design and load variables that denotes the expected number of axle load applications to a terminal serviceability index.
- $\log \rho$ = 7.35 log (D+1) - 0.06
- D = slab thickness, inches
- 4.5 = mean initial serviceability value of all sections at the Road Test
- p_t = terminal serviceability.

In the 1986 and 1993 AASHTO Design Guides, the PCC pavement design model is given as:

$$\log W_{18} = Z_R\, S_o + 7.35 \log (D+1) - 0.06 + \frac{\log\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624*10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32p_t) \log\left(\frac{S_c' C_d (D^{0.75} - 1.132)}{215.63\, J\, (D^{0.75} - \frac{18.42}{(E/k)^{0.25}})}\right) \tag{2.5}$$

Where:

ΔPSI	=	loss of serviceability (p_i - p_t)
D	=	thickness of PCC pavement, inches
S'_c	=	modulus of rupture of concrete, psi
Cd	=	drainage coefficient
Ec	=	elastic modulus of concrete, psi
k	=	modulus of subgrade reaction, psi/inch
J	=	joint load transfer coefficient
W_{18}	=	cumulative 18-kip ESALs at end of time t
p_i	=	initial serviceability
p_t	=	terminal serviceability

For this evaluation, the reliability factor is set to 50% (Z_R = 0). In order to evaluate the AASHTO design equations, it was necessary to ensure that the traffic, climatic, material and other design input variables were determined by the same procedures specified in the AASHTO Guide or used at the Road Test. This was accomplished to the extent possible.

The AASHTO design model was used differently for this analysis than it would typically be used in design. In design, the engineer determines the design slab thickness based on the forecasted traffic over the design life and on a specific loss in serviceability (ΔPSI). In this analysis, the thickness, ΔPSI, and other variables for a specific section were known and the predicted cumulative KESALs were calculated. The ΔPSI was calculated as the difference between the initial serviceability and the serviceability at the time of distress and roughness measurements. An estimate of the traffic carried from the time the pavement was opened to traffic to the time of survey is also known for each test section. If the AASHTO design equation is to be considered adequate and accurate, its predictions of the ESALs to reach the PSI loss should approximate the cumulative ESALs estimated by the State Highway Agencies.

Five sets of analyses were performed individually for GPS Experiments 3, 4, and 5 to examine the ability of the equations to predict the amount of traffic actually sustained by each test section. Initially, analyses were conducted on all available data for each experiment. Then the data sets for each pavement type (JPCP, JRCP, and CRCP) were further separated by environmental regions. Analyses were then performed for each of the four environmental regions for each of the pavement types.

The predicted KESALs were plotted against the estimated KESALs on scattergrams to visually examine the scatter of the data. If the AASHTO model should predict the estimated KESALs exactly, then all of the data would fall on the line of equality shown in each figure.

The results were also presented using bar graphs showing the ratio of predicted to actual KESALs. If the predicted to actual KESALs ratio is less than 1.0, then the AASHTO equation can be said to be conservative. If the ratio is greater than 1.0, the predicted

KESAL capacity of the pavement is greater than the actual KESALs carried to cause the specified loss in PSI, and the equation would produce an inadequate design (at the 50% reliability level).

The resulting plot of predicted vs. actual KESALs using the original AASHTO model (Equation 2.4), appears in Figure 2.9 for JPCP. If the predictions were unbiased for all regions, there would be approximately 50 percent of the points on each side of the line of equality. CRCP was not included in this evaluation of the original AASHTO model because the Road Test did not include CRCP. It can be seen that the original AASHTO model overpredicts KESALs for a majority of test sections (78% of JPCP and 82% of JRCP). Similar scatter plots were developed for separate environmental regions.

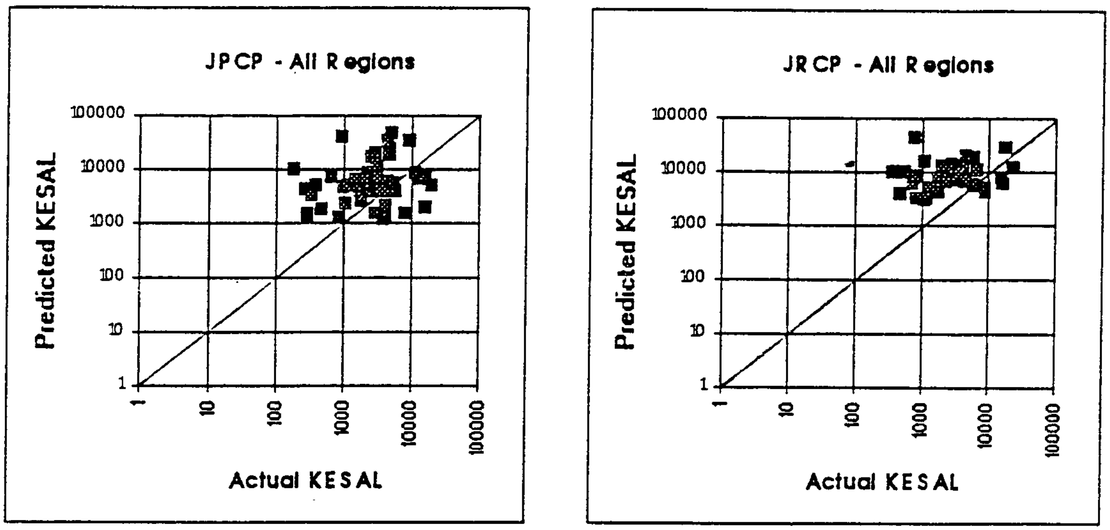

Figure 2.9 Predicted KESALs vs. Actual KESALs for JPCP and JRCP Using Original AASHTO Prediction Model

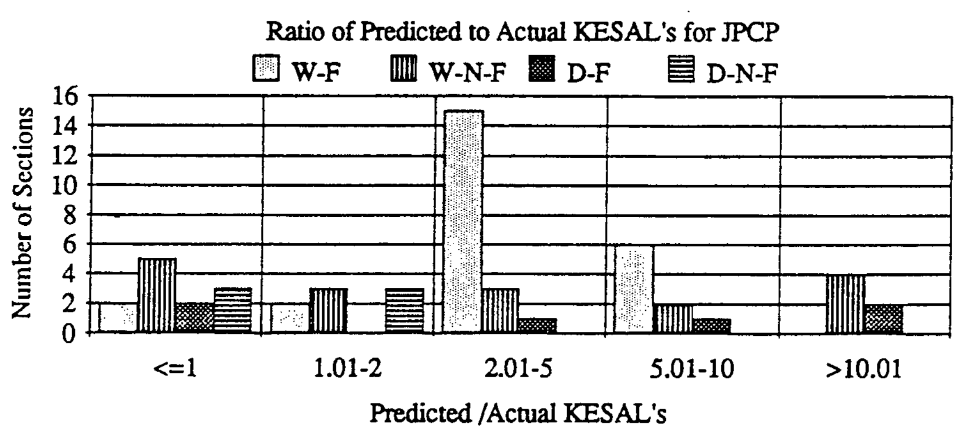

Figure 2.10 Ratio of Predicted KESALs vs. Actual KESALs for JPCP Using Original AASHTO Prediction Model

The distributions of the ratios of predicted to actual KESALs for JPCP and JRCP sections, using the original AASHTO equation, are shown in Figures 2.10. The original AASHTO equation was developed based on the data from a wet-freeze region (Illinois). The results from this analysis, using data from the wet freeze region only, shows that the AASHTO model overpredicts KESALs for 92% of the JPCP sections.

The predicted vs. actual KESALs plots for JPCP, using the 1986 (or 1993) AASHTO model, are shown in Figure 2.11. The plots for individual environmental regions are shown in Figures 2.12 and 2.13. It can be seen that the 1993 model predicted much better than the original AASHTO model for these analysis data sets, suggesting that the addition of several design factors considerably improved the performance prediction capability of the model. However, there are large amounts of scatter about the lines of

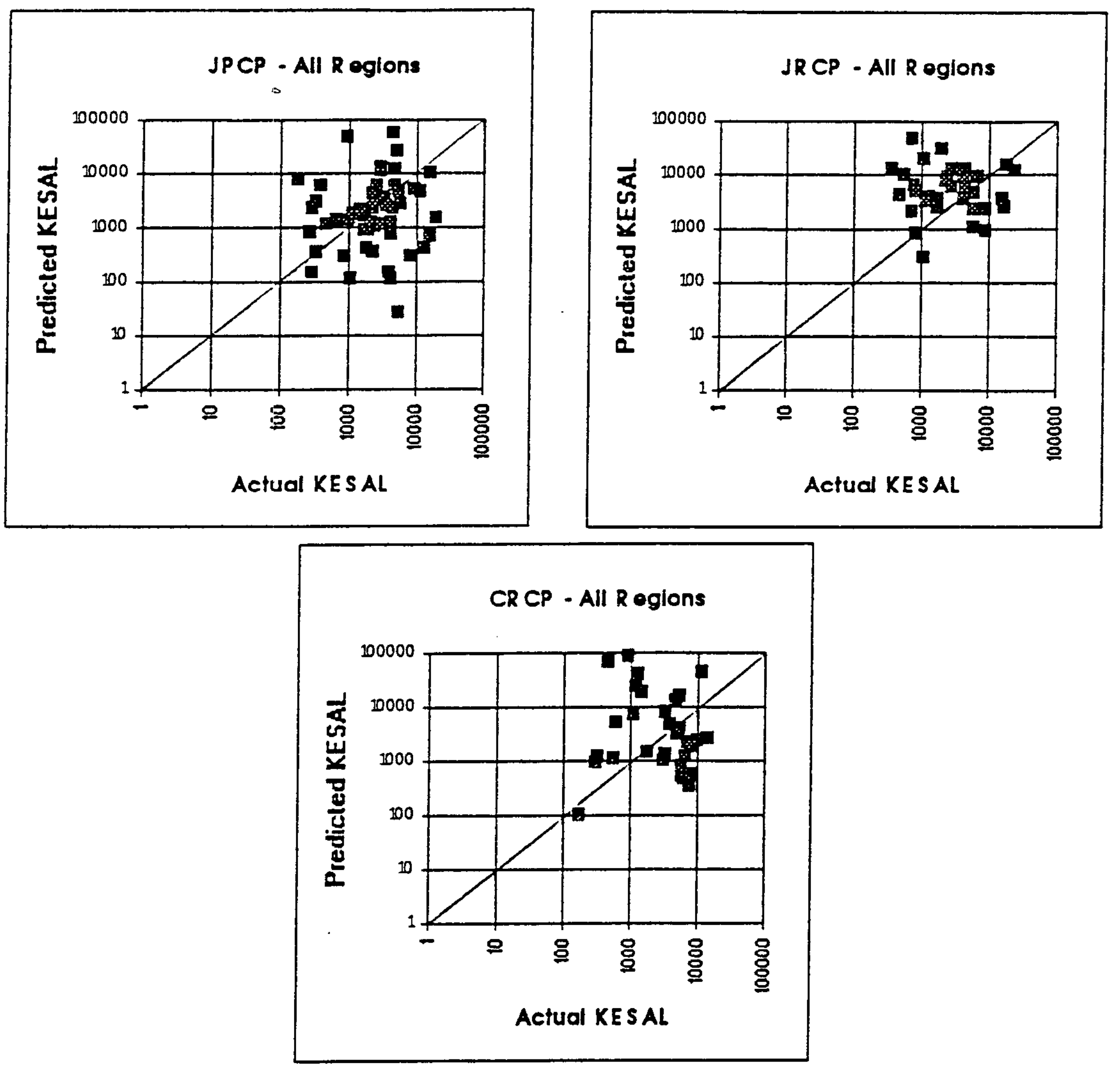

Figure 2.11. Predicted KESAL's vs. Actual KESAL's for JPCP, JRCP, and CRCP Using 1993 AASHTO Prediction Model

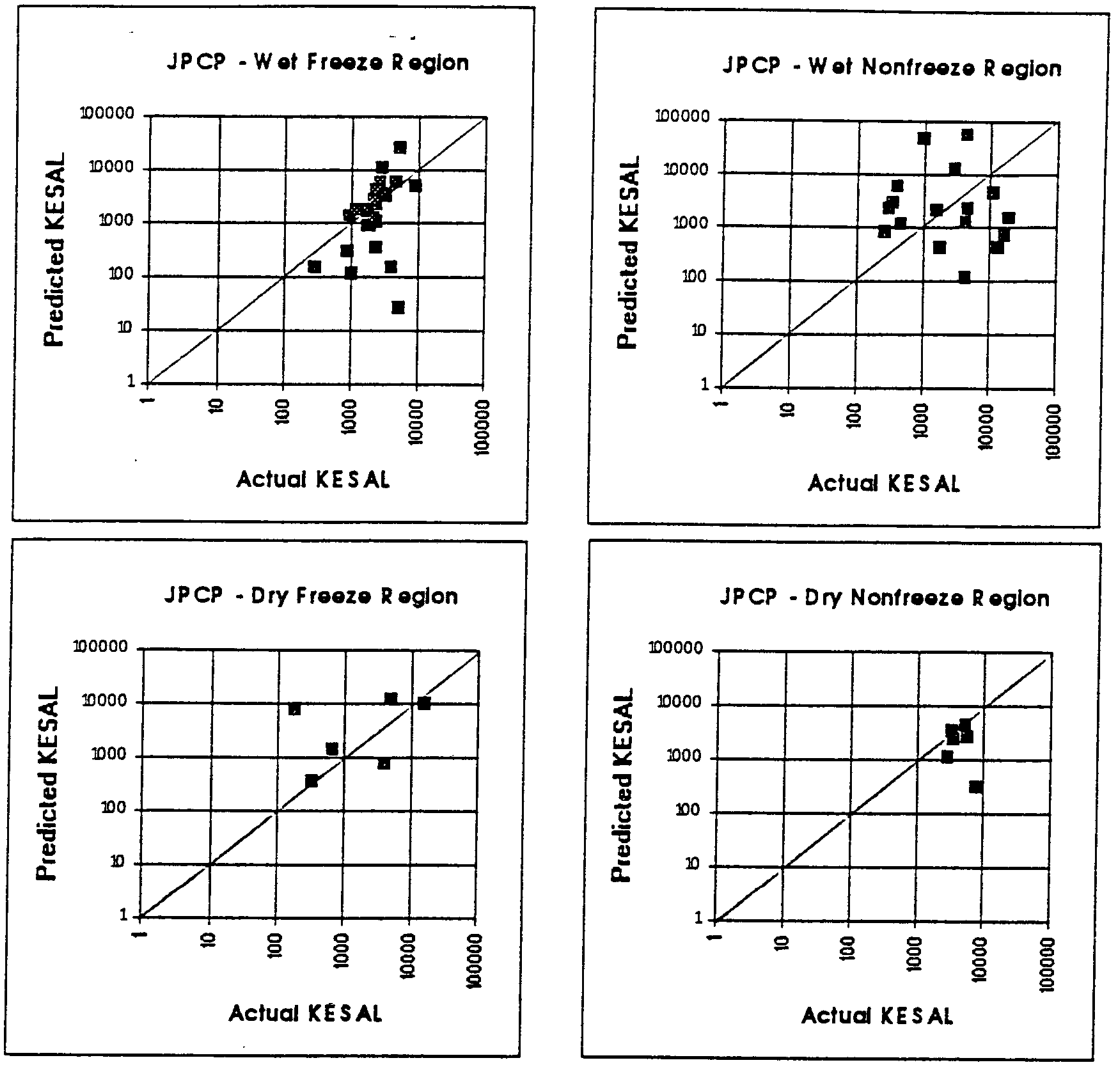

Figure 2.12. Predicted KESAL's vs. Actual KESAL's for JPCP Using 1993 AASHTO Prediction Model.

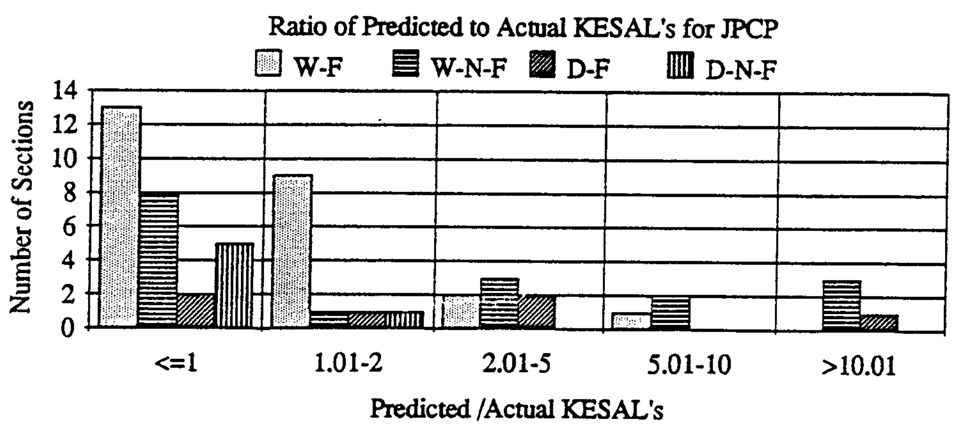

Figure 2.13. Ratio of Predicted KESAL's vs. Actual KESAL's for JPCP Using 1993 AASHTO Prediction Model.

equality, even on these log-log plots. This scatter may be due to several causes, including inadequacies in the model, errors in the inputs, and random performance variations (or pure error). Similar plots were prepared and evaluated for JPCP and CRCP pavements.

In order to analytically determine the ability of the AASHTO concrete pavement design model to predict the actual KESALs observed for the pavement sections, a statistical procedure was followed which determines whether two sample data sets (actual and predicted) are from the same population. The paired-difference method, using the student t-distribution, was used to determine if the KESALs as predicted by the AASHTO equation are statistically from the same population as the actual measured KESALs.

It was observed that t-calc is greater than t-table for one half the data sets when using the original AASHTO model, which indicates that the original AASHTO model is not a reliable predictor of the ESALs actually sustained by the pavement sections. However, for the 1993 AASHTO model, the results show that the null hypothesis is not rejected. This holds true for all climatic regions. These results show that the improvements to the original AASHTO model were beneficial in increasing the accuracy of the design equation.

The initial PSI for all of the pavement sections used to develop the original AASHTO equation was set to 4.5; whereas, in this analysis, the mean estimated initial PSI was set at 4.25. This 0.25 PSI loss reduction caused a reduction in predicted KESALs by the model. Therefore, an analysis using an initial PSI of 4.5 (same as the original AASHO Road Test) was carried out and the statistical results evaluated. The results from this analysis showed that the model does not adequately predict performance for JPCP, JRCP, or CRCP, as it generally overpredicts numbers of axle loads. The fact that for many test sections the calculated current serviceability was greater than the SHAs estimated mean 4.25 initial serviceability suggests that the 4.25 estimate of initial PSI is low. Sections appearing to have negative PSI loss could not be included in the analysis.

Another comparison was made by comparing the actual KESALs to the predicted KESALs at a particular level of design reliability. Thus, the mean $\log W_{50\%}$ prediction is reduced by $Z_R S_o$ (where Z_R = 1.64 for 95% reliability and S_o = 0.35). The predicted (at 95% reliability) versus actual KESALs were plotted (see Figure 2.14). Here, most of the points are below the line of equality, indicating that the consideration of design reliability definitely results in a large proportion of sections (77%) having a conservative design, which is desired. A statistical test was also conducted as before.

The results of these studies were then summarized. The 1986 (or 1993) model appeared to be an unbiased predictor of KESALs. However, a closer examination of the results showed a large scatter of the data about the line of equality that points to possible deficiencies in the model and the inputs used. Thus, even though collectively the adjustments to the 1993 model seem to have improved prediction capabilities in comparison to the original AASHTO model, the evaluation points to the need for further improvements to increase the accuracy of the predictions.

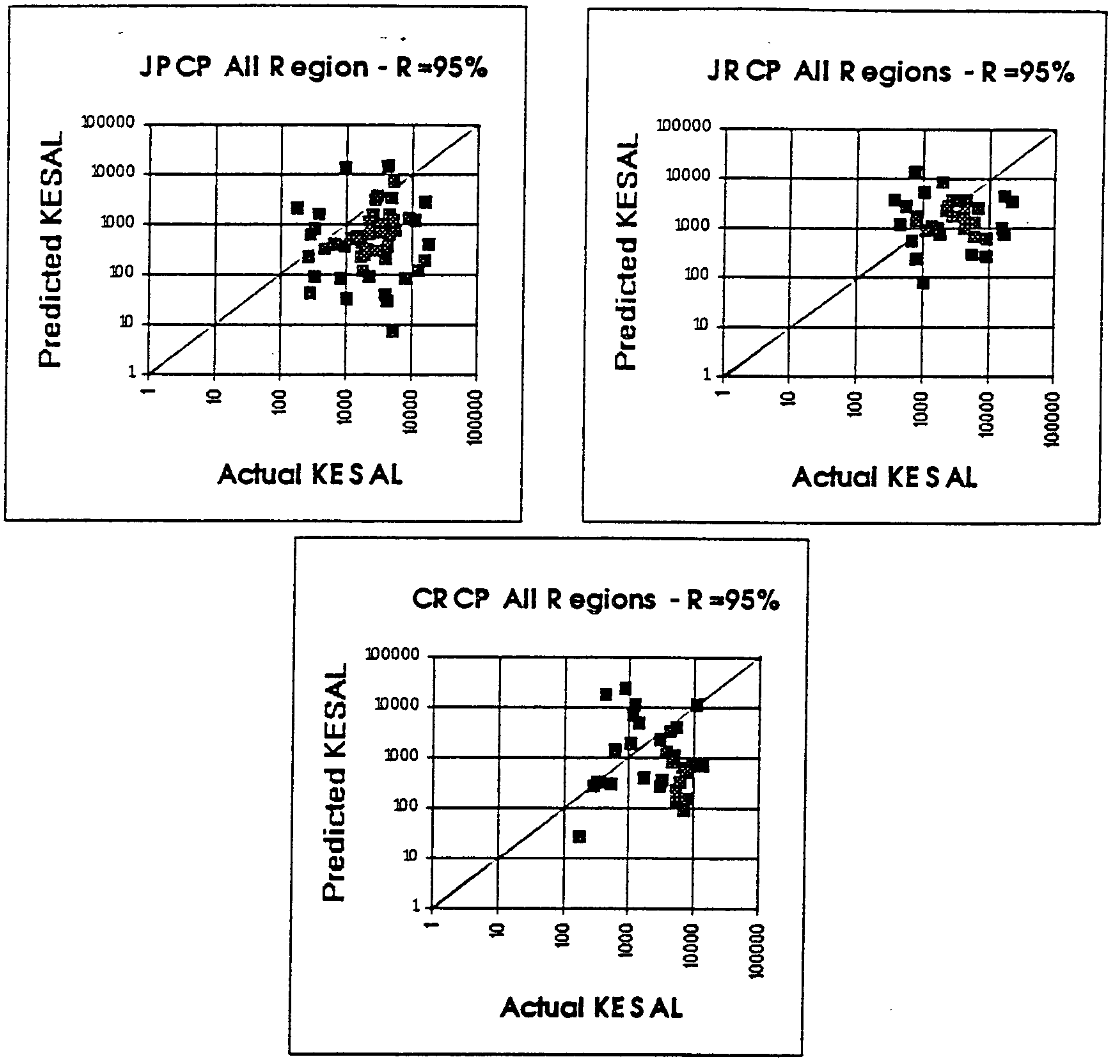

Figure 2.14. Predicted KESAL's vs. Actual KESAL's for JPCP, JRCP, and CRCP Using 1993 AASHTO Prediction Model with 95% Design Reliability.

Procedures Used to Develop Improved PCC Pavement Design Equations

The approach followed for improving the 1993 PCC pavement design equation was essentially identical to that described previously for HMAC pavements. It consisted basically of supplementing the current design equation with equations derived for the sensitivity analyses reported in Volume 3. Specifically, the approach hinged on the use of IRI and distress prediction models as pavement design checks, to ensure that the structural thickness developed with the AASHTO design procedure will meet established performance standards. The following equations were developed to predict key distresses as shown for each of the types of pavements shown:

Joint Faulting:	JPCP non-doweled joint model JPCP/JRCP doweled joint model
Transverse Cracking:	JPCP model (all severities) JRCP model (medium/high severities)
Joint Spalling:	JPCP model (all severities) JRCP model (all severities)
IRI Roughness:	JPCP doweled joint model JPCP non-doweled joint model JRCP model CRCP model

Although the confidence levels for these prediction equations are limited until future improvements may be made when more time sequence data is available, this approach provides the framework for future evolution of design approaches that will consider all significant distresses, instead of one composite index that basically only represents roughness. An example using these equations to check a pavement design appears in Volume 4.

Limitations Imposed by PCC Database Shortcomings

The limitations for the evaluation and improvement of the PCC pavement design equation are those already described as limitations for the sensitivity analyses and the similar studies of the HMAC pavement design equations.

Procedures for Evaluation of the 1993 AASHTO Overlay Design Equations

Procedures Used for Evaluation

The 1993 revisions to the AASHTO overlay design procedure were intended to provide overlay thicknesses that address a pavement with a structural deficiency. A structural deficiency arises from any conditions that adversely affect the load-carrying capability of the pavement structure. These include inadequate thickness, as well as cracking, distortion, and disintegration.

The AASHTO pavement overlay design procedures are based on the concept that time and traffic loading reduce a pavement's ability to carry loads. An overlay is designed to increase the pavement's ability to carry loads over a future design period. The required structural capacity for a PCC or HMAC pavement to successfully carry future traffic is calculated, using the appropriate AASHTO 1993 new pavement design equation. The effective structural capacity of the existing pavement is evaluated using procedures for overlay design presented in the Guide. These procedures can be based on visual survey and material testing results, or the remaining life of the pavement in terms of the traffic that can be carried, or by nondestructive testing (NDT) of the existing pavement,

An overlay is then designed based on the structural deficiency represented by the difference between the future structural capacity required for the future traffic, and the effective structural capacity of the existing pavement. It is obvious that the required overlay structural capacity can be correct only if the future structural capacity and the effective structural capacity are correct. Therefore, it is important to use the AASHTO rigid and flexible design equations properly to determine the future structural capacity, and to use the appropriate evaluation methods to determine the effective structural capacity of the existing pavement.

LTPP data from GPS-6A, GPS 6-B, GPS-7A, GPS-7B, and GPS-9 were used to evaluate the 1993 version of the AASHTO overlay design equations. While data on design life and levels of reliability sought were not available, a limited set of test sections were identified that had sufficient data to support limited evaluations. These included 9 sections with HMAC overlays of HMAC, 5 with HMAC overlays of PCC, and 6 with unbonded PCC overlays of PCC. Even for these test sections, it was necessary to use existing data to estimate values for some of the inputs to the design equations, and procedures used for estimating specific input values are described.

The design equations were then used to predict overlay thicknesses required, and these thicknesses were compared to the thicknesses of the overlays actually constructed. The results from recent profile measurements and distress surveys were also used to evaluate the adequacy of the AASHTO design equation for establishing an appropriate design overlay thickness. A summary of the results from these comparative evaluations were furnished (see Table 2.4).

Although these evaluations were seriously constrained by data limitations, the equation appears for this small data set of 5 test sections to be working quite well for AC overlays of PCC. The evaluations were generally inconclusive for AC overlays of AC and unbonded PCC overlays of PCC.

It is hoped that data will be available in the future as to the design periods and levels of reliability used for design of overlays to be used for comparative evaluations. Conclusive evaluations are probably not possible without this information so that the comparisons may be made on the same design basis.

Limitations Imposed by Data Shortcomings

In all, the LTPP data base consists of 60 GPS-6A, 30 GPS-6B, 33 GPS-7A, 15 GPS-7B, and 28 GPS-9 pavement sections with overlays located throughout the United States and Canada. A variety of information and data have been collected for each section including climatic data, materials properties, traffic loads, profile, and distress data.

Table 2.4 Results from Comparative Evaluation of 1993 AASHTO Overlay Equations

Test Section Number	Type of Pavement	Results From Comparisons			
		Conservative	Adequate	Inadequate	Inconclusive
016012	AC/AC		X		
016109	"				X
351002	"				X
356033	"				X
356401	"		X		
486079	"		@95%		
486086	"		Reliability		X
486160	"	X			
486179	"				X
Subtotals for AC/AC:		1	3	0	5
087035	AC/PCC		X		
175453	"		X		
283097	"		X		
287012	"		X		
467049	"			X	
Subtotals for AC/PCC:		0	4	1	0
69049	PCC/PCC			X	
89019	"				X
89020	"				X
269029	"				X
269030	"				X
489167	"				X
Subtotals for PCC/PCC:		0	0	1	5
Overall Subtotals:		1	7	2	10

These data were to be used in the evaluation of the AASHTO overlay design procedures in this study. However, at the time of the evaluation, the overlay layer data required for this analysis were found to be unavailable for most of the newly overlaid GPS-6B and 7B sections. Similarly, some of the data required for the evaluation of the other pavement types were also not available.

In the end, only nine AC overlays of AC pavement, five AC overlays of PCC pavement, and six unbonded PCC overlays of PCC pavement sections had enough data for the evaluation of the AASHTO overlay procedures. The input data required to compute the future and effective pavement structural capacities for these pavement sections, using the AASHTO design procedures, were obtained from the LTPP data base. Each of the data elements used in the analysis of the GPS-6, 7, and 9 sections are described below:

1. Initial and terminal serviceabilities were not available. Initial serviceabilities were assumed to be 4.2 for HMAC and 4.5 for PCC, and terminal serviceability was assumed to be 2.5.

2. Current overlay serviceability values were assumed on the basis of a recent relationship between International Roughness Index (IRI) and the Present Serviceability Rating (PSR) (8).

3. Future 18-kip ESALs for the design periods were unknown, so the ERES research staff used the SHA estimates of cumulative ESALs and a 6 percent growth rate to predict future ESALs.

4. Subgrade moduli of subgrade reaction "k" had to be estimated in some cases from deflection data after overlay.

5. Data were not available for the "overall standard deviations S_0", which accounts for variability associated with design and construction. Values of 0.49 and 0.39 were assumed for HMAC and PCC models, respectively.

6. Pavement condition adjustment factors were unknown, so values had to be estimated and assumed.

7. Pavement and subgrade resilient moduli were not available, so values were backcalculated from deflection data, assuming a reference temperature of 68F and adjusted to a laboratory result using AASHTO guidelines.

8. PCC flexural strengths were not available, so they were estimated from splitting tensile strengths.

9. Where deflection measurements were taken after the HMAC overlay of a jointed PCC pavement was in place, load transfer coefficients "J" were not available for the original pavement. Recommended load transfer coefficients were taken from Table 2.6 of the 1993 AASHTO Design Guide.

10. The reliability levels that were used in the actual designs for each of the overlays were not available. These were estimated after a sensitivity analysis was performed at different reliability levels.

It is unlikely that much of the data missing will be available for future studies, except that laboratory-derived resilient moduli will be available for each pavement layer.

Summary

The procedures used for the analyses described in detail in Volumes 3 and 4 have been described, and the limitations imposed by database shortcomings have been identified. Discussions of future databases for analyses and possible alternatives procedures will be discussed in subsequent chapters.

3

Data Base Expectations for Future Analyses

It has been well established in previous volumes of this report, and in the previous chapter, that one of the primary limitations of these early analyses has been data shortcomings. Consequently, it is important to discuss what will improve and what will not for the data available to future analysts. These are discussed in this chapter.

Current GPS Data Limitations Not Expected to Change

While the LTPP Data Base for the General Pavement Studies will be considerably enriched by the addition of time-sequence data for distress and performance measurements, and from the site-specific measurements of axle loads, some data elements that are missing now will continue to be missing, and others that are based on best estimates will not be improved. These data elements are identified and discussed below:

- Estimates of initial Pavement Serviceability Indices (PSI) and Roughness (IRI) - As few State Highway Agencies measured roughness on new highways, those data were not available and those distresses included within the PSI equations had yet to occur. Consequently, because the State Highway Agencies were more familiar with their pavements and their construction practices, they were asked to provide estimates for initial PSI. Some State Highway Agencies made serious attempts to do this as accurately as possible, and others simply used the initial value of 4.2 for flexible pavements and 4.5 for rigid pavements established at the AASHO Road Test. At any rate, the estimates that were provided are now and will be in the future the only data available. For flexible pavements, estimates of initial roughness (IRI) were obtained from SHA estimates of the initial PSI, using an equation from Reference 11 (see Chapter 7, Volume 3). It is important to note that neither initial PSI or initial roughness appear in the LTPP data base. The estimated values of initial PSI used for the evaluation of the AASHTO Flexible Design Equation appear in Appendix A of Volume 4. The estimated values for GPS-3, 4, 5, 6, 7, and 9 appear in Appendix B of this report. For rigid pavements, the mean of all the estimates of initial PSI was used as the initial PSI for all sections.

- Missing inventory data - There are quite a few missing items of inventory data, that data from SHA project files that concern the design and construction of the pavements. Some data elements were available for all of the test sections, while others for some test sections were not known and could not be found. Some examples of data elements missing from some test sections include asphalt viscosity, densities of the layer materials in place, gradations of PCC aggregates, moisture-density data from density tests of unbound materials, flexural strength of PCC, and many others. Where such data were missing from the SHA project files, there is no other source practically available for pursuit of these values. Missing data did not always mean that a test section could not be included in the analyses, but it often did, and those test sections omitted appear in Volume 2 with the type of missing data indicated.

- Time sequence distress data prior to commencement of monitoring - There will never be a complete set of time sequence distress data for test sections that were inservice when monitoring began. This simply means that information on the initial occurrence of distress and its early development will be missing for such test sections, but does not mean that their data cannot contribute to the analyses. Time sequence data may be helpful in backcasting some of this data. IRI measurements over time may make it possible to backcast to obtain the initial IRI, if the rate of change is small.

- Accurate values of cumulative ESALs prior to commencement of monitoring -While estimates of annual ESALs will be available in the data base, these estimates are generally considered to be fairly rough. Unless methods are developed to "back cast", using new site-specific data, the estimates of cumulative ESALs before traffic monitoring began will be all that is available to future analysts. However, the magnitudes of the errors should be reduced over time because of the relatively precise measurements to be made in the future.

- Initial stiffness of HMAC - It is well known that the stiffness (or resilient modulus) of HMAC materials generally increases over time due to "hardening" of the asphalt, so it is not possible to consider the stiffness in the analysis on a common basis, as the degree of hardening will vary with age, environment, void ratios, and bitumen characteristics. The only stiffness data that will generally be available are the results from resilient modulus testing on aged HMAC sampled from the pavement. There is really nothing that can be done about this unless a dependable relationship can be developed for aged HMAC mixes.

- Creep compliance for HMAC - Although creep compliance testing was considered to be quite important by many experts, it was omitted from the GPS Materials Test Program due to funding restrictions. Therefore, future predictive equations for rutting will not have this test for GPS sections.

- Pavement condition prior to overlay - Prior to the funding of the SHRP Program, the overlaid test sections (GPS-6, GPS-7, and GPS-9) were recruited with recognition that pavement condition prior to the overlay would not be available. It was subsequently decided to recruit pavements that were planned for overlay, so that the pavement condition prior to overlay would be known. The GPS-6A, 7A, and GPS-9 test section will always be limited in this respect; however, those included in GPS-6B and GPS-7B (for which data on prior condition is available) will eventually support analyses to develop predictive equations for overlaid pavements (they are presently too new to display appreciable distress). As the latter set of test sections is fairly limited, there may be some justification for seeking more test sections that are scheduled for overlay.

- Percent voids in mineral aggregate (VMA) - While this is a fairly important characteristic of an HMAC mix, the bulk specific gravity of the aggregate is required for its calculation. This is generally missing from the inventory data and will not be available from material testing for GPS test sections. The fine aggregates from extractions have been sent to the National Aggregate Association for bulk specific gravities, but this will not be sufficient for calculation of VMA (as the bulk specific gravities for the coarse and fine aggregates combined are needed).

- The 28-day concrete slab flexural strength is not generally available in the data base. The values used in these analyses were estimated from the current core strength and age of the pavement.

Although the data shortcomings listed above will not change, it should be remembered that these shortcomings do not generally apply to the Specific Pavement Studies (SPS), except for the overlay studies where some of the materials and construction data for the existing pavements may be missing. The materials testing program for the SPS is much more detailed and should offer all the materials data required. These test sections will also be monitored closely over their entire life.

Expected Future Data Base Improvements

The GPS data that may not be expected to improve in time was discussed in the last subsection. Those improvements to the GPS data base, with respect to analysis, that may be expected are identified and discussed below:

- Time Sequence Distress Data - periodic collection of distress data will be continuous well into the next century, so information on the occurrence and development of distress for individual test sections will be available for future analyses. This should result in much more robust predictive equations. (Although not strictly a distress, roughness will be monitored and the advantages discussed above will accrue for it also).

- Time Sequence Traffic Data - periodic collection (continuous for some test sections) of site - specific vehicle classification and axle load data will over time increase the precision of the cumulative ESAL estimates. As stated in the previous subsection, the cumulative ESAL estimates available for the early analyses may not be improved, but the data subsequent to the onset of traffic monitoring should be very reliable.

- Materials Data - the lack of layer moduli for these early analyses thwarted the intent of forming mechanistic clusters for use in the multiple regressions for HMAC pavements. This data will be available to future analysts, both in the form of laboratory test results and in the form of backcalculated layer moduli. As results from a recent evaluation of the relationship between laboratory derived and backcalculated moduli generally indicated that they displayed limited correlation, additional studies are needed to select which moduli (or some other) best represents the layer stiffnesses of in-service pavements. The thermal coefficient of expansion is being measured on core samples by the FHWA. The availability of values for this important variable will assist in stress calculations for future evaluations.

- Seasonal Effects - while it is well-known that layer stiffnesses and the occurrence of distresses (including roughness) are dependent on climatic characteristics, as well as traffic and age, this could only be generally considered in these early analyses. Weather data was carefully interpolated from nearby weather stations and were generally believed to reasonably reflect conditions at the test sections. However the seasonal monitoring effort for 64 test sections in the United States and Canada (just getting underway in the fall of 1993) will include monthly monitoring of subsurface moisture and temperatures, rainfall, wind velocities, and measured deflections by a Falling Weight Deflectometer. Quarterly measurements of surface profile will also be made. It is expected that the knowledge gained from these studies will extrapolate to other test sections not monitored seasonally, so that measured deflections can be extrapolated to other seasons throughout a year and to other years. This will allow for consideration of backcalculated moduli on a common basis.

- More test sections displaying distress - distressed test sections were in limited supply for these early analyses, and this seriously limited the inference spaces available for the analyses. The exceptions were rutting in HMAC pavements, faulting in PCC pavements, and roughness for both types of pavements; although the magnitudes for these distresses were generally small. As time passes, more pavements will display distresses and the magnitudes of the distresses will increase. More robust equations should result from these broader inference spaces.

- Increased resolution for distress reductions from photographic negatives - comparisons of the distress measurements from manual (visual) surveys and those from photographic reduction indicated that low severity cracks were frequently not discernable from the photographs. In fact, there were quite a lot of HMAC test sections with low-severity fatigue cracking that were not identified in the data available (only 18 were reported) which might have allowed studies of fatigue cracking. Resolution of differences between results from manual surveys and photographic reduction (including higher resolution equipment for use in the reductions) should result in higher future precision in identification of distresses and their magnitudes and severities.

- Radar measurements of layer thicknesses - ground penetrating radar (GPR) can provide sufficiently precise measurements of layer thicknesses to allow improved results from backcalculation of layer moduli, which are quite dependent on relative precision in estimating layer thicknesses. This will also allow consideration of variability in layer thicknesses as an independent variable. The FHWA plans to undertake a limited effort (around 100 test sections) initially to test out the potential from GPR. If it proves to be effective, perhaps the rest of the GPS test sections could be measured.

- Data that has undergone thorough QA/QC checks - the analyses were delayed over a year because much of the data simply were not available, and some (such as layer moduli) were never available in time for the analyses, which had to be completed in time for SHRP to publish the results. Most of the data used for the early analyses had not undergone QA/QC checks as the procedures and software for the checks had not been completed. However, virtually all of the data for future analyses, will have been checked out and modified where necessary.

Data from the Specific Pavement Studies (SPS) were not available for these early analyses, but will be available for those in the future. However, the additional "explanation" that they may add to the GPS data will be limited for some years until these pavements begin to experience distress.

In general, the data from the SPS projects will be of much higher quality than those from the GPS. The experiments are designed and the projects built to satisfy the designs. A plethora of data collected during construction will obviate the shortcomings in inventory data experienced for GPS. All traffic data will be site-specific, some projects will have in-pavement instrumentation and some will have their own weather stations. The materials testing program is much more thorough than that for the GPS, and the tests will be conducted using protocols that have been fully tested themselves. Complete time sequence distress data will be available.

The inference spaces for the GPS data base can eventually be greatly expanded through the addition of SPS data, which will likely fill in many gaps and expand the inference spaces of the GPS data bases. However, the SPS projects were generally designed to support analyses individually, so they also offer opportunities for analyses of data from designed experiments, with virtually no gaps in the data available for use.

The expectation is that a number of very useful analyses not performed for or included in those early analyses can be conducted during the period from 1994 through 1996, using current data and that from continued monitoring. By 1997, there should be sufficient time sequence data and distressed pavements to allow substantial improvements in predictive equations and their applications to sensitivity analyses, design, and pavement management.

After the turn of the century, most of the pavements should be experiencing distress and long-term time-sequence monitoring data will be available, so it should be possible to harvest the "crop" of information about in-service pavement performance whose limits motivated the LTPP initiative during the 1980's. At that time, there should be adequate data in the GPS data bases to support development and evaluation of new design and overlay models.

4

Alternative Analytical Procedures for Developing Equations to Predict Pavement Distresses

The development of distress prediction models in these analyses was constrained by the late arrival of data, the need for statistically linear models for the sensitivity analyses, funding, and the necessity of completing the work so SHRP could publish it before SHRP expires. Although not required by the contract, the project staff had hoped to utilize pavement response data (strains, stresses, and deflections) from the Contract P-020B studies for HMAC pavements at Michigan State, and to utilize this response data with layer moduli and other data elements with nonlinear regression techniques to produce mechanistic-empirical models of higher quality than could be produced with these constraints. Unfortunately, this could not be done, but it should be carried forth at the earliest opportunity. An illustrative analysis was accomplished for fatigue cracking of PCC pavements using backcalculated moduli and calculated stress in the slab.

While the predictive equations developed are considered to be generally reasonable, they were very considerably constrained by the lack of test sections with time-sequence distress data. There is no doubt that better models can be developed in the future, becoming ever more robust as the monitoring continues. Data will also be available for evaluating and improving mechanistic-empirical models that exist and others that may be developed. The procedures for collecting data and conducting quality checks are still evolving, so the quality of data should also improve with time and additional study.

One objective of this research was to develop and test out methods for conducting sensitivity analyses, and these results appear in Volume 3. While these procedures appear to be workable, additional study and work may improve these procedures as well. However, this is not the subject of this chapter and will not be discussed further.

The procedures used for the analyses reported were reviewed and discussed periodically by a Data Analysis Working Group, an Expert Task Group on Experimental Design and Analysis, the Pavement Performance Advisory Committee, SHRP staff, and SHRP Contract P-001 Technical Support Staff. In addition to comments and guidance provided while the analyses were in progress, there were a number of suggestions offered during that period for alternative approaches to developing predictive equations. These are identified and discussed below.

Discriminant Analysis

Discriminant Analysis can be used to identify the variables that affect whether or not a pavement will undergo distress (in particular cracking). This procedure is uniquely different from the other linear analyses simply because it aids in determining whether or not a pavement will crack as opposed to how much it will crack.

Probabilistic Failure-Time Models to Include Unobserved Distress Data

Paterson applied an estimation procedure (See Appendix B of Reference 11) based on the principles of failure-time analysis to include both stochastic and censored data in the development of predictive models. This approach requires multiple distress surveys during a "window of observation", with the "censored data" relating to test sections that: 1) had experienced distress at a level defined as "failure" prior to the onset of monitoring (left censoring) and 2) test sections which still had not experienced distress to a failure level when last surveyed (right censoring). The procedure uses the statistical method of maximum likelihood estimation to take advantage of both observed data and censored data.

Survival Analysis

A survival curve analysis could be conducted for a "group" of similar pavements. For example, a "group" could be defined as JPCP (GPS-3) with no dowels, wet-freeze climate, and a slab thickness less than 25 cm thickness. This would require that most of the sections in that "group" had reached some level of failure (such as IRI greater than 125 in/mi).

The result of the survival analysis would be a survival curve showing percent sections either failed or survived versus age or ESALs. Curves like this could be derived for many different "groups" of pavements in the LTPP database. This type of relationship is very useful in communicating pavement performance to management for example.

Neural Network Approach

The Neural Network Approach allows the model to learn the data trends that can be used in iterative fashion to develop better and better models. This process would require continuing time sequence data over a considerable time. This is a relatively new procedure that is only beginning to gain attention for pavement engineering applications.

Bayesian Analysis

Bayesian Analysis techniques involve conducting multiple regressions on observed data, but also including other information (such as other data, other predictive models, and experience from experts) converted to simple numerical forms (called "priors") as additional data. Some of these "priors" are not easy to obtain and transform to numerical data and generally will not be as reliable as LTPP data. The primary application appears to be to supplement data bases where limited observed data are available, and those where long-term data is not available. The most obvious application is the case where the number of test sections is very limited, as in the application for the Canadian LTPP.

Nonlinear Regression Analysis

Nonlinear regression techniques are commonly used for developing empirical models, but were not used for these analyses because of the statistical linearity requirements for the sensitivity analyses and lack of time and funding. These techniques should certainly be used for developing future predictive equations for use in design and/or pavement management. As layer moduli will be available, equation forms for trial may be developed to reflect available knowledge of the mechanics of the materials and the pavement structure.

Advanced Modern Regression Techniques

Many ingenious iterative regression techniques in the area of "robust" outlier detection, "nonparametric" variable transformation, error variance stabilization, and inclusion of variable interactions have been developed and have gradually gained popularity.

A "robust" outlier detection technique provides a more reliable and objective way for data-screening. The advantages of nonparametric regression techniques, which do not need to explicitly specify an unjustified predictive model form, offer the maximum flexibility in model specifications and the best fit to the data. Some of these techniques include the least median squared regression (12), alternating conditional expectations (13), additivity and variance stabilization (14), and projection pursuit regression (15).

The main features of these techniques are briefly summarized in Table 4.1. Several of these techniques were used in the analysis of the rigid pavement data and the development of prediction models.

Table 4.1 Main Features of the Modern Regression Techniques Introduced

Algorithms	Main Features
Least Median Squared (LMS or "Robust") Regression	A robust regression technique, extremely powerful in outliers detection
Alternating Conditional Expectations (ACE or "Expectation")	A nonparametric regression technique, providing optimal variable transformations to maximize the squared multiple correlation (R^2)
Additivity and Variance Stabilization (AVAS or "Stabilization")	A nonparametric regression technique, transforming both sides of the additive model to achieve constant error variance assumption
Projection Pursuit Regression (PPR or "Projection")	A nonparametric regression technique, capable of modeling variable interactions

Mechanistic Based Models

The underlying mechanisms are generally understood for some of the distress types, such as fatigue cracking for flexible and rigid pavements, rutting of flexible pavements, and faulting of jointed concrete pavements. Extensive research studies have determined the general mechanisms and functional relationships for these distresses. For these and any other distress types where the mechanics of their development are generally understood, it is recommended that this knowledge be utilized in the development of mechanistic based predictive models. The appendices to Volume 3 contain two documents, one related to flexible pavements and the other to rigid pavements, that describe some of the basic concepts of mechanistic modeling. The use of dimensional analysis has also been found to be helpful in identifying clusters of variables that relate strongly to the distress under consideration.

The authors feel that this approach to development of predictive models from LTPP data may yield the most useful models for use in pavement design and management. Several of the above listed statistical modeling tools can be extremely useful in this endeavor, including the non-linear regression and the advanced modern regression techniques.

Alternative Approach Proposed for Modeling Roughness

One very experienced reviewer felt that the modeling approach to the roughness equation was fundamentally wrong, and that was the reason for the problems encountered in the use of the HMAC pavement roughness model for design purposes. His experience indicated that the rate of roughness progression should be initially slow and increase with time, whereas the rates for models developed from LTPP data were initially large and then diminished over time. The research staff agrees that the form suggested does occur, but the form used is most representative for the LTPP data.

This may be a consequence of comparing data for highways in "third-world countries" to those for relatively high-quality highways in the United States and Canada. There are substantial differences in pavement structures and in the incidences of illegal overloads. Another possible explanation is that the roughness measured on LTPP test sections was closely related to rutting, which clearly follows the equation form used.

The reviewer felt that better models would have resulted from structuring the model to estimate slope rather than the magnitude of the change in roughness. The model proposed was:

$$R_t = e^M[R_o + A^B N] \tag{4.1}$$

where:

- R_t = roughness at time t
- R_o = initial roughness
- M = Environmental zone * age
- B = Constant
- N = Cumulative ESALs

For the sensitivity analyses, the requirement for statistical linearity would necessitate simplification of the equation form to:

$$[R_t - R_o / N] = e^M A^B C^D \tag{4.2}$$

where:

- A and C = Structural explanatory variables
- B and D = Coefficients

This opinion and the proposed models are furnished so that they may be considered by future analysts.

5

Evaluation of Design Procedures

The purpose of this chapter is to bring the experience of the research staff in evaluating the AASHTO pavement design equations to bear on recommendations for future evaluations of design models in general. As rapid changes are being made in mix designs for bound materials and quality control during construction, the design methods should ideally be sufficiently flexible to reflect these improvements in future performance predictions. While such ideal models may not be attainable until well into the 21st century, it appears useful to first consider the characteristics of an ideal design model before discussing how to evaluate existing and future design procedures. Recommendations for evaluating design procedures are then discussed.

Qualities of an Ideal Design Procedure

As pavement designs have future highway service as their objective, they ideally should be capable of accurately predicting the future performance of pavements in terms of pavement structure, material properties, joints and reinforcement, expected traffic loading, drainage, and the environment in which the pavement is to function. Future performance should be estimated in terms of individual distresses, their amounts, and their severities (including roughness and friction loss as distresses). Most (if not all) current design procedures have their basis in past experience, but ideal design procedures should be capable enough to reflect the effects from a variety of initiatives presently underway.

The initiatives that are expected by the research staff to have the greatest impacts on future performance of pavements are material-related, drainage-related and design-related; although improvements in truck suspensions, tire design, and truck design can offer important impacts on performance also. These initiatives are discussed below, followed by a discussion of the impacts of not having ideal models that can directly consider their impact.

Mix Designs Related to Performance

Mix design procedures during most of the twentieth century have been based on materials tests specifically aimed at material characteristics that were known to be generally beneficial to the performance of pavements. This has been necessary because adequate mathematical relationships between measurable material properties and the performance of in-service pavements did not exist. However, recent research initiatives by the NCHRP, SHRP, FHWA, research organizations in other countries, and others have been aimed at development and use of materials tests that correlate closely to specific measurements of performance such as fatigue cracking, permanent deformation durability problems, and low temperature cracking. These mix design procedures are generally configured to optimize a mixture such that it will minimize the predicted future occurrence of the distresses expected to be significant to performance. Tests have also been developed to evaluate the durability of materials for climatic conditions. There is every reason to expect that the resulting mixtures (HMAC, PCC, aggregates, etc.) will carry more traffic than their predecessors before experiencing levels of distress requiring intervention.

The ideal design procedure should then utilize predictive equations that are capable of reflecting the changes to performance that may be expected in the future. However, empirical equations based on performance of pavements in the past will be tied to the past, unless they include explanatory variables that can "explain" the modifications to the mix.

Proliferation of Modifiers and Additives

A broad range of modifiers have been developed in recent years and are now available to modify HMAC and PCC mixture properties. For pavements with modified mixes, the same set of potential problems exist for predicting earlier damage than may actually occur, as for the performance-based mixtures described above. New mix design procedures under development are generally configured to deal with the modified cements and mixtures.

Performance-Based Specifications

Another initiative is building momentum to replace "recipe-type" and even end-result type specifications with "performance-based" specifications where key material and construction characteristics correlate with the performance of the pavement. This leads to the ability to provide rational incentive pay adjustments for increased levels of quality of construction and decrease variability in both layer thicknesses and material properties. Predictive equations will be needed to provide predictions of damage, for pavements built with performance specifications.

Impacts From Less Than Ideal Predictive Equations

It is fortunate that the anticipated results from using predictive equations derived from past pavement performance are expected to predict more damage than should actually occur when the improved pavement materials are realized. This means that such equations, when used in design procedures, should result in conservative designs, and can be "calibrated" later as data accrues for future pavements.

The use of mechanics to model pavement behavior appears to offer more potential for an ideal model than empirical procedures. This has been an elusive goal of many researchers for decades. These efforts have been quite successful for "response models", which predict deflections and strains (and hence stresses) in a pavement structure; using layer thicknesses, layer moduli, and Poisson's ratio to characterize the structure, and surface pressures to simulate axle loads. However, the successes in applying mechanics to predict distress (such as cracking, permanent deformations, and roughness) have been very limited, and such models generally require some empirical material coefficients in addition to the predicted responses. While some progress is being made toward more mechanistic distress models, perhaps the only hope in the immediate future of applying these potentially more ideal, and certainly more logical, models effectively is to use data from LTPP pavements to "calibrate" them. This can be done by revision of input values on some logical basis, or by applying "calibration functions" to the predicted results.

The "bottom line" for impacts from inability to develop "ideal models" appears to be that we are accustomed to applying available knowledge through design procedures that are both simplistic and far from ideal, so the opportunities for incremental improvements in predictive models should represent progress to the highway community. After all, functional pavements have been designed and have provided reasonable service for most of this century. The goals are to provide better service at reduced costs, and the "mechanisms" are in place for these gains. It is only necessary to actively and patiently pursue the opportunities for improvements.

Procedures Used for Evaluation of the AASHTO Design Equations

The various techniques that were used to evaluate the AASHTO design equations appear in Table 5.1. The most obvious, and perhaps the most powerful one, is to compare the ESALs predicted by the AASHTO equations to result in the loss of PSI measured on the pavements, to the actual (in this case estimated) ESALs experienced by the pavement. This technique was used for evaluation of both the flexible and rigid design equations. After the results from this comparison became available, the gross differences for the flexible pavements and the less dramatic differences for the rigid pavements led the research staff to the evaluation techniques discussed below.

Table 5.1 Design Equation Evaluation Techniques Used by Research Staff

Design Equation Evaluation Techniques	HMAC Pavement	PCC Pavement	Overlay
1. Comparisons of predicted to actual ESALs	X	X	
2. Linear regressions on LTPP data using AASHTO equation form	X		
3. Comparisons of inference spaces for AASHO Road Test and the LTPP Data Base	X		
4. Factorial Study of AASHTO Equation Predictions of ESALs	X		
5. Regressions on ratios of predicted to Observed ESALs to Identify additional variables needed	X		
6. Study of effects of differences between backcalculated layer moduli and those from laboratory testing	X		
7. Comparisons of predicted ESALs for initial PSI values of 4.5 and 4.25		X	
8. Comparison of Predicted ESALs at 50% and 95% Reliability Levels		X	
9. Paired-difference method, using the student t-distribution, to ascertain if the ESALs predicted by the AASHTO equation are from the same population as the estimated ESALs		X	
10. Comparison of thickness of constructed overlay to the thickness required by the AASHTO overlay design procedure			X

Evaluation Techniques Used for the Flexible Pavement Design Equation

The results from the comparisons of predicted to estimated ESALs were so pronounced, that the research staff felt that the emphasis needed to be placed on finding out why that was the case. This led to the individual studies identified as numbers 2 through 6 in Table 5.1. Technique 2 made it clear that the AASHTO design equation form was not at all suited to the LTPP data. Technique 3 explored whether the AASHTO design equation would become adequate as the inference space was reduced toward the inference space that actually existed for the road test. The equation still did not appear to be adequate, even for the reduced inference spaces.

Technique 4 was conducted simply to study predictions for a broad range of input values. This was strictly an analytical exercise with no direct relation to the data in the LTPP data base. This resulted in the identification of portions of the inference space (some portions within the inference space for the AASHO road test) that predicted ridiculous values for PSI loss, some ranging up to billions of PSI loss.

One means of graphing the differences between the predicted and "observed" ESALs was to plot histograms of the ratios of predicted to observed. This led to the use of multiple regressions to predict the ratios and to identify indirectly the independent variables that were significant to the loss of PSI. It was found that those included in the AASHTO design equation were in fact significant, but that 2 other variables (average annual rainfall and average annual number of days below freezing) were also significant. As data far outside the inference space for the road test was included, it was not surprising that other environmental "explanation" would be required for reasonable predictions for the LTPP data.

As the subgrade moduli used for the development of the predictive equation from the road test were assumed to be 3,000 PSI, and the subgrade moduli from the backcalculations using the seventh sensor were much greater than that, it appeared to be advantageous to try the equation using resilient moduli from laboratory testing. Although that data had not been available at the time that most of the analyses were underway, results had been received for 106 of the subgrades late in the analysis period. Therefore, these values were used in the design equation and new comparisons made. It was found that the design equation still had serious problems, but the predictions were much more accurate when laboratory resilient moduli were used.

All six of these techniques proved very informational for these early analyses.

Evaluation Techniques Used for the Rigid Pavement Design Equation

The results from the comparisons of predicted to "observed" ESALs for PCC pavements did not result in quite the dramatic results as for the flexible pavements. Consequently, the techniques used for additional evaluations by the ERES research staff were numbers 7, 8, and 9 in Table 5.1. Technique 7 explored the effects of using the mean value of initial PSI estimated by the State Highway Agencies to use of the value of 4.5 from the road test. These comparisons indicated that this difference contributed to the differences in the comparisons of predicted ESALs to those estimated by the State Highway Agencies.

For direct comparisons, it is necessary to set the reliability level at 50% for the design equations. As this is less than would usually be selected for design, Technique 8 was used to compare the results at a 50% and at a 95% reliability level. This resulted in recognition that most designs would be conservative at the 95% reliability level.

The research staff then used Technique 9 to see if the predicted ESALs and the estimated ESALs were from the same population. This resulted in the conclusion that the ESALs from the original AASHTO equation were not from the same population, but that those from the 1993 equation were. This indicated that the numerous "improvements" made to the design equation over the years had in fact improved it.

Each of these techniques were very helpful in the evaluation of the AASHTO rigid pavement design equation.

Evaluation Technique Used for the Overlay Design Equations

Much of the data required as inputs by the overlay design equations were not available, so estimates were made. Only one type of comparison really offered meaningful evaluations. Technique 10 was utilized for this, and was very similar in effect to Technique 1. Because of the data limitations and the limited number of test sections for which sufficient data were available to utilize even this limited approach, the evaluations were found to be inconclusive for AC overlays of AC and unbonded PCC overlays of PCC. However, the limited data indicated that the design procedure was working well for AC overlays of PCC. The limited results obtained were not caused by the technique applied, but because the data available were so limited.

Recommendations for Future Evaluations of Design Equations

Technique 1 in Table 5.1, comparisons of predicted to actual ESALs, should be the first technique applied. If the differences from the comparison are consistently minor, then the evaluation may be complete. In the more likely case when the differences are not minor, any one of Techniques 2 through 9 may offer additional information of use in the evaluation. In fact, those techniques used to evaluate the flexible design equation and those used to evaluate the rigid design equation might be profitably utilized on either. The selections of the techniques utilized should depend on the nature of the predictive equation itself and on the results of comparisons of predicted to actual ESALs (or whatever other comparisons are made, such as overlay thicknesses in Technique 10).

The future evaluations should focus on different climatic zones, since the AASHTO design equation has little ability to adjust to different climates. Future design equations should either be capable of taking climate into account directly, or individual equations should be used for different climatic zones.

6

Summary

The experience gained by the research staff for these early analyses has been shared with future researchers and analysts in this volume. The data analysis procedures employed for these early analyses have been discussed, the shortcomings of the early LTPP data base and the expected improvements for future analysts have been identified, alternative procedures for developing predictive models have been identified for future consideration, and procedures for evaluating design methods have been proposed. It is hoped that these discussions will simplify analytical work requirements for future analysts of the LTPP data.

As it would be repetitious to go into further detail here, the reader is directed to the "Executive Summary" at the front of this volume for a broader discussion of the contents.

References

1. Hocking, R.R. and O.J. Pendleton, "The Regression Dilemma-Variable Elimination, Coefficient shrinkage and Robust Estimation", Communications in Statistics, Vol. 12, No. 5, 1983.

2. Freund, R.J. and R.C. Littell, "SAS® System for Regression, SAS® Series in Statistical Applications", Second Edition, SAS® Institute Inc., SAS® Campus Drive, Cary, North Carolina 27513, Pages 123-124.

3. Statistical Sciences, Inc., "S-PLUS User's Manual," Seattle, Washington, March 1993.

4. "AASHTO Guide for Design of Pavement Structures," AASHTO Task Force on Pavements (1986).

5. HRB Special Report 61E, "The AASHTO Road Test, Report 5, Pavement Research," National Research Council, Washington, D.C., 1962.

6. HRB Special Report 73, "The AASHTO Road Test", Proceedings of a Conference Held May 16-18, 1962, St. Louis, MO, 1962.

7. "Microsoft EXCEL™ Version 4.0 User's Guide". Microsoft Corporation, 1992.

8. "AASHTO Guide for Design of Pavement Structures, 1993". American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

9. Al-Omari, Bashar and M.I. Darter, "Relationships Between IRI and PSR", Transportation Engineering Series No. 69, University of Illinois, Urbana, IL, 61801.

10. S.H. Carpenter. Selecting AASHTO Drainage Coefficient. Presented at the 67th Annual Meeting of the Transportation Research Board, January 1989.

11. Paterson, William D.O., "Road Deterioration and Maintenance Effects, Models for Planning and Management", published for the World Bank by the John Hopkins University Press, Baltimore and London, 1987.

12. Rousseeuw, P.J., "Least Median of Squares Regression", Journal of the American Statistical Association, Vol. 79, pp. 871-880, 1984.

13. Breiman, L. and J.H. Friedman, "Estimating Optimal Transformations for Multiple Regression and Correlation", Journal of the American Statistical Association, Vol. 80, pp. 580-619, 1985.

14. Tibshirani, R., "Estimating Transformations for Regression via Additivity and Variance Stabilization", Journal of the American Statistical Association, Vol. 83, pp. 394-405, 1987.

15. Frieman, J.H. and W. Stuetzle, "Projection Pursuit Regression", Journal of the American Statistical Association, Vol. 76, pp. 817-823, 1981.

Appendix A

"White Paper" on Biases Resulting From Restrictions on GPS Project Selections

POTENTIAL POPULATION BIAS RESULTING FROM RESTRICTIONS ON PROJECT SELECTION

FOR

GENERAL PAVEMENT STUDIES, LONG-TERM PAVEMENT PERFORMANCE STRATEGIC HIGHWAY RESEARCH PROGRAM

The purpose for this paper is to identify issues related to potential bias that could result from restrictions established on candidate projects for General Pavement Studies (GPS). Such an allegation has already been made formally by the American Trucking Associations, Inc. (see Appendix A), and other such allegations from other special interest groups may be forthcoming. It is appropriate that we pause now and review our reasons for restricting the populations and evaluate the potential effects to see if unanticipated and undesirable biases will occur as alleged. In doing so, we must keep in mind that almost every decision on experiment design that we make affects the population to be sampled, and that tradeoffs must be made to optimize the resulting data base for its primary role as a basis for predictive equations.

The specific list of considerations provided to the State Highway Agencies in "Instructions for Selecting Candidate GPS Projects and Filling of Data Forms" follow:

1. Choose candidate projects such that steep grades, sharp corners, roadway structures (i.e. bridges or large culverts), unusually deep cuts or high embankments, or other unusual geometry is avoided.

2. Attempt to select projects with varying ages.

    a. Constructed or rehabilitated in 1970 or later.
    b. Distribution of ages from young to old.

3. Length of candidate projects should be at least one mile (or perhaps as little as 1500 feet if part of a previous or ongoing performance monitoring program).

4. Should not consider projects for which the outside lanes have been added or widened. Also, projects that have undergone surface milling before overlay should not be considered.

5. Should not consider projects currently exhibiting, or with the potential for, serious studded tire damage.

6. Include projects with high ESAL/year where possible as many states will have few or no such projects; however, projects should not be considered for which safety considerations would preclude closing lanes for coring and boring and deflection testing.

7. Where feasible, candidate projects for any experimental cell should be distributed geographically around the state.

8. Do not select projects with abnormal conditions such as severely expansive subgrades (i.e., P.I. greater than 40), subgrades subject to more than nominal frost heave, and unique surface or subsurface drainage problems. (Where material that is not frost susceptible has been placed such that the frost does not penetrate the frost susceptible material, the project may be considered as a candidate.)

9. Projects should experience uniform traffic movement over entire length (i.e. no major interchanges or change in number of lanes).

10. Do not consider concrete pavement sections with D-cracking or reactive aggregates or asphalt concrete pavements with severe stripping problems (contemporary specifications preclude or limit occurrence of these distresses).

11. Do not consider concrete pavements with Uni-tube joint inserts or mechanical load transfer devices other than round dowels. (The experiments also include projects without dowels.)

12. <u>Do not consider pavements with fabric interlayers, or with seal coats currently less than one year in age.</u>

13. Open-graded base materials that are free draining appear to have great potential for use under rigid pavements. Try to include a few such projects if they exist in your state and otherwise meet the requirements for a candidate project.

14. If project is part of another pavement monitoring program, please attach a brief discussion of the types of information being collected, length of time, etc.

15. Do not consider pavements with rigid surfaces that have been ground as this changes roughness and masks other distresses. Longitudinal grooving that does not change profile or materially affect recognition of distress is allowable.

Review of the considerations listed above will indicate that some are indeed restrictive as to the population of pavements to be included, while others (specifically items 6, 7, 13, and 14) suggest characteristics that are desirable or provide guidelines. Those that have the potential for introducing bias will be considered individually below.

Perhaps the best way to approach this study of potential bias will be to consider each of the items listed above separately. Discussions of these individual items (using the same item numbers) appear below.

1. The primary reason for the decision to avoid steep grades, sharp corners, roadway structures, unusually deep cuts or high embankments, or other unusual geometry was to avoid potential effects on performance that might result from unusual features. While these effects are of interest and could be studied separately through other funding, they represent only a small percentage of the total roadway miles and could bias the resulting predictive equations when they are applied in the future to the usual case of roadways without these unusual features. On balance, it appeared much more desirable to maintain representation for the general case.

2. The restriction to projects that had been constructed or rebuilt in 1970 or later effectively eliminated a large population of pavements constructed in the 1960's and earlier. This decision was reached after careful consideration of the pros and cons because:

   a. There is a potential for undesirable bias from studying pavements whose design and material specifications do not reflect contemporary practice, so it appeared to be better not to include pavements that were more than 15 years old.

b. The older a pavement is, the more difficult it is to search out the required historical data.

c. We specifically did not want pavements that had been overlaid more than once because it would be very difficult to sort out statistically the relative effects from each overlay. Therefore, many of the older pavements would have been eliminated if overlaid more than once anyway, and those with overlays older than 15 years would fall out of the program much more rapidly than desired.

d. This is scheduled as a 20-year program, so there is an advantage to starting with newer pavements for long-term observation.

3. Requiring that the length of candidate projects be at least one mile in length is really not very restrictive as most highway projects are much longer than that. The concept was that a minimum length of one mile would allow much more flexibility in random selection of a test section within the project, ~~that would be most representative for the experiment~~.

4. The elimination of projects for which the outside lanes had been added or widened was a decision to maintain representation (as for Item one) for the majority of pavements. The same applies to pavements that have undergone surface milling before overlay as such projects are obviously not representative of the general population.

5. The decision to avoid projects that were exhibiting, or had the potential for, serious studded tire damage was a practical one, because it is virtually im-

possible to discriminate between the studded tire damage and rutting.

8. The decision to exclude projects having severely expansive subgrades, more than nominal differential frost heave, or unique surface or subsurface drainage problems was again a practical one as such abnormal condition would tend to bias the performance of the pavements and mask the occurrence of other distresses or performance measures. The ATA alleges that such exclusions, without similar exclusions of load induced distresses, will unbalance the GPS portion of the project. In reality, this does not restrict the environmental effects very much as the effects of expansive subgrades with little to high volume change potential are still included, as are projects with subgrades subject to frost heave at nominal differential levels. Consequently, the environmental effects are there, but severe levels that would tend to mask out or bias the effects of other distresses are excluded. It appears that the allegations by the ATA primarily reflect differences in desired outputs from the LTPP studies. The ATA is highly interested in cost allocation aspects and thus the effects of truck loading versus environment on pavement performance. Most of the rest of us are interested in improved cost allocations, but our primary emphasis is on predictive equations to support pavement management and design. This decision had nothing to do with any "sacred cows" as implied by the ATA letter, nor is it necessarily true that it will tend to bias the ATA position relative to effects of truck loading, as the interactions of the roughness with the dynamic loads of trucks might exacerbate the effects due to trucks.

9. Eliminating those portions of projects with non-uniform traffic movement over the length of the project is a practical matter to avoid serious bias to the data base.

10. The restriction against concrete pavements with D-cracking or reactive aggregates or asphalt pavements with severe stripping problems reflects the opinion that the extensive studies that have been conducted to control these problems through appropriate material specification will bear fruit; thus we do not want to bias the data base with performance of such pavements when there is strong reason to suspect that they will be minimal in the future. Also, the occurrence of stripping in asphalt concrete pavements generally manifests itself in accelerated occurrence of other distress types such as rutting, so serious biases would result. The comments in item 8 relative to the interests of the ATA apply here as well.

11. The exclusion of concrete pavements with uni-tube joint inserts or mechanical load transfer devices other than round dowels is again to maintain representation of the great majority of concrete pavements. Uni-tube joint inserts and various other devices have been tried in the past, but were neither used broad-scale nor generally accepted, so round dowels are believed to be most representative of the population desired in the data base.

12. The decision not to use fabric interlayers is again for representation of the population as a whole. Fabric interlayers are frequently used, and the effects of fabric interlayers on performance could be the subject of specific studies, but should not be

allowed to bias the results from the General Pavement Studies. The restriction of seal coats that are less than one year in age at the time of candidate project selection represents a compromise to allow seal coats, but to have at least 2 years prior to data collection for distresses in the original pavement to start manifesting themselves again. It would be preferable to have pavements without the seal coats, but this would represent a severe restriction on the population.

15. The restriction against pavements with rigid surfaces that have been ground is a practical one because grinding tends to smooth out the profile and thus change the roughness measurements, and also tends to mask other distresses. As longitudinal grooving does not change profile or materially affect recognition of distress, it was allowed.

In summary, review of the restrictions that were applied in the General Pavement Studies Instructions do bias the population, but generally amount to deliberate efforts to select a population from the whole which is most representative and which will produce the most useful predictive equations for of t he majority of pavements and of current construction practices pavement performance. In any experiment, there are decisions to be made as to the population that is desirable in the experiment and any such decision results in some form of bias. It appears that the population selected is appropriate for the overall aims of the LTPP studies, but may not be totally consistent with the ambitions of various special interest groups with relation to the studies.

APPENDIX A

ATA LETTER, APRIL 10, 1986

AMERICAN TRUCKING ASSOCIATIONS, INC.

2200 Mill Road, Alexandria, VA 22314

POLICY DIVISION

DEPARTMENT OF HIGHWAY POLICY
John L. Reith
Director

Richard A. Lill
Assistant Director
and Highway Engineer
(703) 838-1788

April 10, 1986

Mr. L. G. Byrd, Interim Director
Strategic Highway Research Program
Suite 343
444 N. Capitol St., N.W.
Washington, D.C. 20001

Dear Gary:

I appreciated the opportunity to sit in on the LTPP workshops as an observer, which I did on April 1 and 2. The sessions were valuable, but they also raised serious concern about the site selection process.

The concern relates to the exclusion of specific environmental and material quality conditions from the pavements which are eligible to be LTPP/GPS test sections. These exclusions, without similiar exclusions of load induced distresses, will unbalance the GPS portion of the project. If implemented, these limitations very much narrow the broad objectives of the LTPP project which the trucking industry has supported.

I have attached pages 15 and 16 of the document titled "The GPS Project Selection Process" where the specific exclusions are contained. The items excluded reflect the following specific pavement performance parameters:

Serious studded tire damage.
Subgrades that are highly expansive (P.I. >40).
Subgrades that are subject to (differential) frost heave.
PCC with D-cracking or reactive aggregates.
AC with severe stripping problems.

I recognize that the site selection process is complicated by the fact that the GPS sections cannot reasonably be expected to be in the exact same condition, as would be the case if only new pavements were in the test. I also recognize that severely abnormal pavement conditions should not be included in the selected sections, as such would not be representative of the system as a whole.

However, the exclusions above have been directed solely to those aspects of highway pavement performance which reflect environmental and material properties, and again emphasize only

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the loading/structural aspects of the pavement problem as did the AASHO Road Test. There is no exclusion, for instance, of a candidate pavement that is pumping or one which is rutted. I believe, as the exclusions are stated, that the selection process unbalances the experiment. In my view, this is contrary to the thrust of the discussions the LTPP pavement advisory committee held and to Mark Goodes' statement that there would be no "sacred cows" in the project.

If only new pavements could be used, then all factors would be on a reasonably equal basis and would reflect the current state of the art with respect to design, materials selection, quality control, etc. In this instance, it could be presumed that all the different forces which might affect pavement performance would be randomly represented.

Since the GPS experiment will include pavements of different ages and since there is a need to have all forces reasonably well represented, I urge that it is necessary to establish certain minimum criteria and conditions which all sections should meet. The criteria could be established to separate abnormal situations from conditions which might be considered to be normal. I think criteria could be established from the aspect of whether the condition is something which shows up in the "short term", as contrasted to the same kind of force whose effects which may accumulate more slowly over the "long term".

I note that specific criteria are given for the exclusion of highly expansive subgrades. I accept the fact that this criteria may very well separate the abnormal from the normal. I believe similiar criteria should be established for all of the elements whose measurement which will ultimately constitute the measures of "performance" in this project, including pumping, faulting, rutting, D-cracking, frost heave, profile, material deficiency, etc. Such criteria are necessary, in my view, to establish that there is a reasonably consistent "base condition" of all the test sections at the start of the monitoring.

I am distributing copies of this letter fairly widely, in order that my concerns reach the appropriate SHRP/LTPP individuals as quickly as possible. I will be pleased to discuss the concerns expressed here at any time.

Sincerely,

Dick

Richard A. Lill

cc: Thomas Larson
Marke Goode
Lowell Bridwell
Rex Leathers
Members LTPP Advisory Committee

CONSIDERATIONS FOR SHA SELECTION OF CANDIDATE PROJECTS

(THIS WILL BE "THE" TEST SECTION)

1. CANDIDATE PROJECTS MUST INCLUDE AT LEAST ONE MILE THAT DOES NOT HAVE:

   - STEEP GRADES, SHARP CORNERS, OTHER UNUSUAL GEOMETRY
   - ROADWAY STRUCTURES (BRIDGES OR LARGE CULVERTS)
   - DEEP CUTS OR HIGH EMBANKMENTS
   - OUTSIDE LANES ADDED OR WIDENED
   - OLD SURFACE MILLED BEFORE OVERLAY
   - SERIOUS STUDDED TIRE DAMAGE
   - SEAL COATS CURRENTLY LESS THAN ONE YEAR OLD
   - SUBGRADES THAT ARE HIGHLY EXPANSIVE (P.I. > 40)
   - SUBGRADES THAT ARE SUBJECT TO DIFFERENTIAL FROST HEAVE
   - MAJOR INTERCHANGES OR CHANGES IN NUMBER OF LANES

"EXCLUDED"

- PCC WITH D-CRACKING OR REACTIVE AGGREGATES
- AC WITH SEVERE STRIPPING PROBLEMS
- JOINTS WITH UNI-TUBE JOINT INSERTS, OR MECHANICAL LOAD TRANSFER DEVICES OTHER THAN ROUND DOWELS
- FABRIC INTERLAYERS
- ORIGINAL PAVEMENTS OR OVERLAYS CONSTRUCTED PRIOR TO 1970

Appendix B

SHA Estimates of Initial Present Serviceability Indices, GPS-3, 4, 5, 6A, 6B, 7A, 7B, and 9 Test Sections

SHA Estimates of Initial Present Serviceability Indices by GPS Test Section

SHRP ID	EXP NO.	INITIAL PSI
13028	3	3.83
13998	5	4.51
14007	4	3.90
14084	4	3.80
14127	6B	L
14129	6B	4.39
15008	5	4.40
16012	6A	4.80
16019	6A	S
21008	6A	3.6
26010	6A	3.6
46053	6A	4.25
46054	6A	3.72
46060	6A	3.74
47079	5	3.77
47613	3	3.12
47614	3	3.5
53011	3	4.20
53059	4	4.20
53073	7B	4.70
53074	7B	4.70
54019	4	4.20
54021	4	4.20
54023	4	4.20
54046	4	4.20
55803	5	4.20
55805	5	4.20
63005	3	4.2
63010	3	4.5
63013	3	4.5
63017	3	4.5
63019	3	4.5
63021	3	4.5
63024	3	4.5
63030	3	4.5

SHRP ID	EXP NO.	INITIAL PSI
63042	3	4.5
66044	6A	4.3
67455	5	4.5
67456	3	4.5
67493	3	4.5
69048	9	4.5
69049	9	4.5
69107	9	4.5
83032	3	4.2
86002	6A	4.2
86013	6A	4.2
87035	7A	4.2
87036	7A	4.2
87776	3	4.2
87783	6A	4.2
89019	9	4.2
89020	9	4.2
94008	4	4
94020	4	4.5
95001	5	4.2
101201	4	4.5
104002	4	4.5
105004	5	4.5
105005	5	4.5
123804	3	4.00
123811	3	4.20
124000	3	4.00
124057	3	4.20
124059	3	4.20
124101	6B	3.80
124109	3	4.40
124135	6B	4.30
124136	6B	4.20
124137	6B	4.20
124138	3	4.10

SHRP ID	EXP NO.	INITIAL PSI
133007	3	4.25
133011	3	S
133015	3	4.46
133016	3	4.45
133017	3	S
133018	3	S
133019	3	4.34
133020	3	4.31
134118	9	S
134119	6B	S
135023	5	S
137028	7A	S
163017	3	
163023	3	
165025	5	
166027	6A	
174074	4	N
174082	4	3.94
175020	5	3.99
175151	7B	N
175217	7B	N
175423	7A	N
175453	7A	4.32
175843	5	4.29
175849	5	N
175854	5	N
175869	5	N
175908	5	N
176050	6A	N
177937	7A	4.67
179267	5	N
179327	7B	N
183002	3	S
183003	3	3.77
183030	3	S

SHRP ID	EXP NO.	INITIAL PSI
183031	3	S
184021	4	S
184042	4	S
185022	5	S
185043	5	S
185518	7B	S
185528	7B	3.91
185538	7B	3.82
186012	6A	S
189020	9	S
193006	3	4.19
193009	3	4
193028	3	4
193033	3	4.03
193055	3	4
195042	5	4.5
195046	5	4.03
196049	6A	4.15
199116	7B	4.1
199126	7B	4.2
201006	6A	S
203013	3	3.79
203015	3	3.79
203060	3	4.06
204016	4	4.25
204052	4	S
204053	4	3.81
204054	4	S
204063	4	N
204067	4	S
206026	6A	S
207072	7A	3.59
207085	7A	S
209037	9	S
213016	3	L

SHRP ID	EXP NO.	INITIAL PSI
214025	4	3.5
216040	6A	L
216043	6A	3.7
224001	4	3.8
233013	3	4.5
233014	3	4.5
237023	7A	4.5
245807	5	N
263068	3	N
263069	3	N
264015	4	N
265363	5	N
266016	6A	N
267072	7A	N
269029	9	N
269030	9	N
273003	3	L
274033	4	L
274034	4	L
274037	4	L
274040	4	L
274050	4	L
274054	4	L
274055	4	L
274082	4	L
275076	7B	L
276064	6A	L
276250	9	L
276300	9	L
277090	7A	N
279075	9	L
282807	6B	4.2
283018	3	4.2
283019	3	4.2
283089	6B	4.2

SHA Estimates of Initial Present Serviceability Indices by GPS Test Section

SHRP ID	EXP NO.	INITIAL PSI
283090	6B	3.9
283091	6B	4.1
283093	6B	4.1
283094	6B	4.1
283097	7A	4.1
283099	7B	4.2
284024	4	4.3
285006	5	4.2
285025	5	4.3
285803	5	4.1
285805	5	4.2
287012	7A	3.9
289030	9	3.9
294031	4	4.5
294036	4	4.5
294069	4	4.5
295000	4	4.5
295047	5	4.5
295058	4	4.5
295081	4	4.5
295091	4	4.5
295393	7B	4.2
295403	6B	4.2
295413	6B	4.2
295473	7B	4.2
295483	7B	4.2
295503	4	4.5
296067	6A	4.2
297054	7A	4.2
297073	7A	4.2
306004	6A	3.5
307075	6A	4.3
313018	3	4
313023	3	3.7
313024	3	3.8

SHRP ID	EXP NO.	INITIAL PSI
313028	3	3.75
313033	3	4.1
314019	4	L
315052	5	L
316700	6B	3.9
316701	9	4.3
316702	7B	4.3
317005	7A	L
317017	7A	L
317040	7A	4.3
317050	7A	S
323010	3	4.31
323013	3	3.64
327084	3	4.5
328139	3	4
344042	4	4.8
346057	6A	4.8
351002	6A	4.31
353010	3	4.75
356033	6A	4.33
356035	6A	4.33
356401	6A	4.33
361008	6B	S
364017	4	4.5
364018	4	4
373008	3	4.4
373011	3	4.4
373013	3	L
373044	3	4.4
373807	3	4.4
373816	3	4.4
375037	5	4.4
375826	5	4.4
375827	5	4.4
383005	3	L

SHRP ID	EXP NO.	INITIAL PSI
383006	3	L
385002	5	L
386004	6A	N
393013	7B	L
393801	3	3.6
394018	4	N
394031	4	N
395003	5	L
395010	7B	S
396019	6	N
397021	7A	
399006	9	S
399007	9	S
399022	9	N
403018	3	4.5
404155	9	4.5
404157	3	4.5
404158	5	4.5
404160	3	4.5
404162	3	4.5
404166	5	4.5
405021	5	4.5
406009	6A	4.2
406010	6A	4.2
407024	7A	4.2
415005	5	5
415006	5	5
415008	5	5
415021	5	5
415022	5	5
416011	6A	5
416012	6A	5
417018	7A	5
417019	7A	5
417025	7A	5

SHRP ID	EXP NO.	INITIAL PSI
417081	5	5
421598	5	4.2
421606	4	4.2
421608	6B	4.5
421610	7B	4.2
421613	7B	4.2
421614	7B	4.2
421617	7B	4.2
421618	6B	4.5
421623	3	4.2
421627	9	4.2
421690	4	4.2
421691	7B	4.2
423044	3	4.2
425020	5	4.2
427025	7A	4.2
427037	7A	4.2
429027	9	4.2
447401	7A	4.5
453012	3	4.5
455014	5	4.5
455017	5	4.3
455034	5	4.3
455035	5	4.5
457019	7A	4.2
463009	3	4.5
463010	3	4.76
463012	3	S
463013	3	4.3
463014	3	N
463052	3	4.6
463053	3	4.36
465020	5	S
465025	5	S
465040	5	N

SHRP ID	EXP NO.	INITIAL PSI
467049	7A	3.8
469106	6B	4.17
469197	6B	4.62
472008	6B	4.8
473108	6B	4.7
473109	6B	4.8
473110	6B	4.8
476015	6A	4.7
476022	6A	4.5
481046	6A	3.50
481093	6B	4.00
481116	6B	L
481119	6B	4.30
483003	3	S
483010	3	S
483569	9	S
483589	3	S
483629	7A	3.50
483699	4	L
483719	5	S
483779	5	S
483845	9	4.10
484142	4	S
484143	4	S
484146	4	L
484152	4	L
485024	5	L
485026	5	N
485035	5	S
485154	5	S
485274	5	S
485278	5	L
485283	5	L
485284	5	4.50
485287	5	L

SHA Estimates of Initial Present Serviceability Indices by GPS T

SHRP ID	EXP NO.	INITIAL PSI
485301	5	L
485310	5	L
485317	5	L
485323	5	S
485328	5	3.50
485334	5	4.20
485335	5	4.50
485336	5	S
486079	6A	4.10
486086	6A	3.50
486160	6A	3.80
486179	6A	N
487165	7A	N
489167	9	4.40
489355	9	4.00
491004	6A	
491005	6A	
491006	6A	
491007	6A	
493010	3	
493011	3	
493015	3	
497082	3	
497083	3	
501681	6B	L
501682	7B	L
501683	6B	L
512564	5	4.8
515008	5	4.3
515009	5	4.3
515010	5	4.2
533011	3	4.8
533013	3	4.8
533014	3	4.8
533019	3	4.8

SHRP ID	EXP NO.	INITIAL PSI
533812	3	4.8
533813	3	4.8
536020	6A	4.5
536048	6A	4.5
536049	6A	4.5
536056	6A	4.5
537322	6A	4.5
537409	3	4.8
544003	4	4.4
544004	4	4
545007	5	4.4
547008	7A	4.9
553008	3	4.6
553009	3	4.8
553010	3	4.2
553012	3	4.1
553014	3	4.2
553015	3	S
553016	3	4.23
553019	3	4.7
555037	5	4.9
555040	5	4.3
556351	3	N
556353	3	3.69
556354	3	S
556355	3	S
557030	7A	N
563027	3	3.9
566029	6A	4.2
566031	6A	4
566032	6A	3.9
826006	6A	4.5
826007	6A	4.5
836450	6B	4.5
836451	6B	S

SHRP ID	EXP NO.	INITIAL PSI
836452	7B	4.45
843803	3	4
846804	6A	4.1
866802	6A	4.2
893001	3	4.4
893002	3	3.8
893015	3	4.4
893016	3	3.7
899018	9	4.6
906400	6A	
906410	6B	S
906412	6B	S
906801	6A	

S — Slope of regression non-negative or back calculated value not between 3.5 & 5.0.

N — No data available for section.

L — Data lacking for sufficient regression/estimate.