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16. Abstract

This report documents the final results of a long-term performance study of CRC pavement sections in Texas. An existing distress-index that describes CRCP deterioration as a function of punchouts and patches was used in conjunction with a CRCP data base spanning over 14 years and over 751 test sections statewide to develop three types of performance prediction models.

The models were calibrated using Survival Analysis, a statistical technique still unexplored in the analysis of pavement performance, that is adequate for estimating the reliability of a design or a device. This approach is theoretically sound, and it gave accurate results. The models permit the remaining life of a test section to be estimated from visual condition survey data.

During the calibration process, the significance of several variables affecting pavement performance was tested. Among these variables were the elastic modulus of the Portland cement concrete and the modulus of reaction on top of the subbase, obtained by back-calculation from deflection data. This back-calculation is generally done by inverse application of layered theory, performed by one of the many computer programs available in the literature for this type of calculation. There seems to be no consensus as to which program yields the best results. In addition, for rigid pavements, the back-calculation can be also done by inverse application of plate theory. The back-calculation phase of this study was used to compare results obtained with layered and plate theory. A significant discrepancy was found, and the possible causes were analyzed.

It is felt that the most important contributions of this study are in the redefinition of the problem of pavement performance models and in the innovative and theoretically sound technique applied to develop the models. The complementary analyses of the problems of restricted inference spaces for and of error propagation in pavement performance studies are also very important, because they draw attention to a crucial limitation of most pavement performance models that is often overlooked.

It is hoped that the models can be useful for CRC pavement management and that the findings of this study can contribute to a better understanding of CRCP deterioratrion. It is felt that the data collection procedures and the data base developed in this study can be useful for many other research studies. It is strongly recommended that future attempts at calibrating a pavement performance with this reliability be made with the statistical approach used in this study.

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by

Angela Jannini Weissmann
B. Frank McCullough
W. R. Hudson

Research Report Number 472-7F

Research Project 3-8-86-472

Rigid Pavement Data Base

conducted for

Texas State Department of Highways and Public Transportation

in cooperation with the

U.S. Department of Transportation Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

August 1989

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily represent the official views or po!icies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actuallly reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

This is the final report of Research Project 3-8-86-472. It describes the development of performance prediction models for Texas continuously reinforced concrete pavements (CRCP), which was the culmination of a research study that had several other objectives, already fulfilled and described in previous reports. A list of these reports is on the next page.

The authors hope that the models will be useful as managerial tools and that the findings of this study will contribute to a better understanding of CRCP deterioration. They feel that the data collection procedures and the data base developed in the last phase of this study will be useful for many other research studies.

We would like to express our gratitude to the staff of the Center for Transportation Research and The University of Texas at Austin Transportation Engineering group, for their helpfulness during this research study.

We would also like to express our gratitude to Dr. Peter John and Dr. David Firth, from the Department of Mathematics of The University of Texas at Austin, and to Dr. Hani Mahmassani, from the Department of Civil Engineering of The University of Texas at Austin, for their valuable suggestions.

LIST OF REPORTS

Research Report 472-1, "Evaluation of Proposed Texas SDHPT Design Standards for CRCP," by Mooncheol Won, B. Frank McCullough, and W. R. Hudson, presents the results of an evaluation of a proposed CRCP Design Standard which considers the various coarse aggregates used in the mix, describes the theoretical models used in the study, and discusses several important design parameters of CRCP. April 1988.

Research Report 472-2, "Development of a Long-Term Monitoring System for Texas CRC Pavement Network," by Chia-pei J. Chou, B. Frank McCullough, W. R. Hudson, and C. L. Saraf, presents the application an experimental design method to develop a long-term monitoring system in Texas. It also discusses the development of a distress index and a decision criteria index for determining the present and terminal conditions of pavements. October 1988.

Research Report 472-3, "A Twenty-Four Year Performance Review of Concrete Pavement Sections Made with Siliceous and Lightweight Coarse Aggregates," by Mooncheol Won, Kenneth Hankins, and B. Frank McCullough, presents the results of statistical analyses over a 24-year performance period of continuously reinforced concrete pavements made with lightweight and conventional/standard aggregates. The performance variables studied include pavement deflections and visual condition survey data. The report also presents recommendations and directions for future research emanating from the study for consideration by CRCP designers. April 1989.

Research Report 472-4, "Development of Procedures for a Statewide Diagnostic Survey on Continuously Reinforced Concrete Pavements," by Angela Jannini Weissmann and Kenneth Hankins, describes and discusses the studies carried out to develop the procedures for collecting the diagnostic data. November 1989.

Research Report 472-5, "A Statewide Diagnostic Survey on Continuously Reinforced Concrete Pavements in Texas," by Angela Jannini Weissmann and Kenneth Hankins, describes the preparations for and conduction of a statewide diagnostic survey of continuously reinforced concrete pavements. It also presents a summary of the data and discusses the results. August 1989.

Research Report 472-6, "A Continuously Reinforced Concrete Pavement Database," by Terry Dossey and Angela Jannini Weissmann, describes the development and the contents of the statewide CRC pavement data base. November 1989.

Research Report 472-7F, "Development of Performance Prediction Models for Continuously Reinforced Concrete Pavements in Texas," by Angela Jannini Weissmann, B. Frank McCullough, and W. R. Hudson, discusses the development of models to predict performance of CRC pavements, using a theoretically sound statistical technique that was still unexplored for modeling pavement performance. August 1989.

ABSTRACT

This report documents the final results of a long-term-performance study of CRC pavement sections in Texas. An existing distress-index that describes CRCP deterioration as a function of punchouts and patches was used in conjunction with a CRCP data base spanning over 14 years and over 751 test sections statewide to develop three types of performance prediction models:

Model 1:

Output = Natural logarithm of the cumulative number of equivalent single axle load applications.

Inputs = Observed distress index differential.
Probability of surviving the traffic repetitions given by the output.

Model 2:

Output = Natural logarithm of the cumulative number of equivalent single axle load applications.

Inputs = Slab thickness.
Observed distress index differential.
Probability of surviving the traffic repetitions given by the output.

Model 2:

Output = Natural logarithm of the cumulative number of equivalent single axle load applications.

Inputs = Load transfer coefficient.
Observed distress index differential.
Probability of surviving the traffic repetitions given by the output.

The models were calibrated using Survival Analysis, a statistical technique still unexplored in the analysis of pavement performance, that is adequate for estimating the reliability of a design or a device. This approach is theoretically sound, and it gave accurate results. The models permit the remaining life of a test section to be estimated from visual condition survey data.

During the calibration process, the significance of several variables affecting pavement performance was tested. Among these variables were the elastic modulus of the Portland cement concrete and the modulus of reaction on top of the subbase, obtained by back-calculation from deflection data. This back-calculation is generally done by inverse application of layered theory, performed by one of the many computer programs available in the literature for this type of calculation. There seems to be no consensus as to which program yields the best results. In addition, for rigid pavements, the back-calculation can be also done by inverse application of plate theory. The back-calculation phase of this study was used to compare results obtained with layered and plate theory. A significant discrepancy was found, and the possible causes were analyzed.

It is felt that the most important contributions of this study are in the redefinition of the problem of pavement performance models and in the innovative and theoretically sound technique applied to develop the models. The complementary analyses of the problems of restricted inference spaces for and of error propagation in pavement performance studies are also very important, because they draw attention to a crucial limitation of most pavement performance models that is often overlooked.

It is hoped that the models can be useful for CRC pavement management and that the findings of this study can contribute to a better understanding of CRCP deterioration. It is felt that the data collection procedures and the data base developed in this study can be useful for many other research studies. It is strongly recommended that future attempts at calibrating a pavement performance with this reliability be made with the statistical approach used in this study.

KEYWORDS: continuously reinforced concrete pavement, nondestructive testing, nondestructive evaluation, back-calculated materials properties, pavement performance, structural evaluation, performance prediction model, reliability.

SUMMARY

This report documents the development of performance prediction models for continuously reinforced concrete pavements (CRCP) in Texas, using the CRCP survey data and the diagnostic data collected and stored in previous phases of this research study.

Chapter 1 provides background information about the research study objectives and the overall research approach. The results of two distinct approaches to backcalculating material characterization parameters from available deflection data is documented in Chapter 2.

A critical review of the data for the models is made, with emphasis on the expected sources of errors these data are subject to.

The problem of predicting the pavement performance is redefined, considering the reliability associated with predictions. The statistical method used to calibrate the models is briefly summarized, and its improved adequacy, compared to traditional regression methods, is discussed. The data reduction for the model is presented and the final calibration process is documented. The results are presented and discussed.

The problem of pavement performance studies is defined and discussed from the point of view of the inference spaces available for statistical analysis. The error propagation in performance models is discussed, using this study as an illustrative example. The concept and the modeling of the reliability of a system are defined and discussed, and the theoretically sound approach used in this study is compared to a traditional approach used in pavement design. Final recommendations for practical implementation of the findings of this research study are made. Suggestions for future research concerning studies about CRCP performance and pavement performance in general are laid out.

IMPLEMENTATION STATEMENT

The models developed in this study can be implemented at managerial levels, where they can be used to determine the remaining life of any given Texas CRCP section, based on condition survey data. The remaining life is useful either as a criterion for the choice between several design options or as a criterion to schedule major maintenance services.

The statistical method used to develop the models is theoretically sound and appropriate for analyzing survival time data and estimating reliability. It is recommended that this method be implemented in future studies of this kind.

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CHAPTER 1. INTRODUCTION

INTRODUCTION

The problem of maintaining and rehabilitating the nation's infrastructure is overwhelming; because of the increasing demands of an already gigantic infrastructure, decisions in these areas must be made frequently and efficiently. Inventory, condition, and performance data are the primary sources for support of those decisions, but, at present, the state-of-the-art is such that techniques for effective data use in managerial decisions are less advanced than those for data collection and storage. This situation causes costly maintenance, overruns of construction projects, and prevents agencies from implementing rational management programs for maintenance and rehabilitation (Ref 3). The challenge posed by these hurdles requires a new perspective, one that integrates knowledge of materials, structural models, data acquisition techniques, and data base management. In the pavement area, this has been termed a Pavement Management System (PMS) (Ref 14). Figure 1.1 summarizes the basic components of an integrated PMS.

In Texas, an effective PMS is vital because the cost of maintaining the Texas network is estimated to exceed $4 million per year. An important part of a PMS is the monitoring of pavement performance by periodically obtaining network-level condition survey data. CRCP survey data have been collected by the Center for Transportation Research (CTR) since 1974, and a considerable amount of information concerning CRCP has accumulated. This information has been used to develop new design and maintenance criteria (Refs 6, 13, 23, 24, 25, 28, 40, 42, and 45). As a part of this effort, techniques for monitoring CRCP and for modeling its performance were developed and applied.

This report concludes the studies of CRCP performance under this project, which used the data to develop performance prediction models for CRCP and reflected the particular conditions affecting Texas CRCP. These models were calibrated using a theoretically sound statistical technique appropriate for estimating the reliability, or survivor, function of a random variable.

BACKGROUND

Pavement deterioration is very complex; it is difficult to obtain accurate data on important relevant variables, such as traffic composition, and the costs of gathering data for studying pavement performance are high. However, the costs of building and maintaining roads are also high, and that has motivated research efforts concerning design and rehabilitation strategies, the findings of which imply considerable and much needed

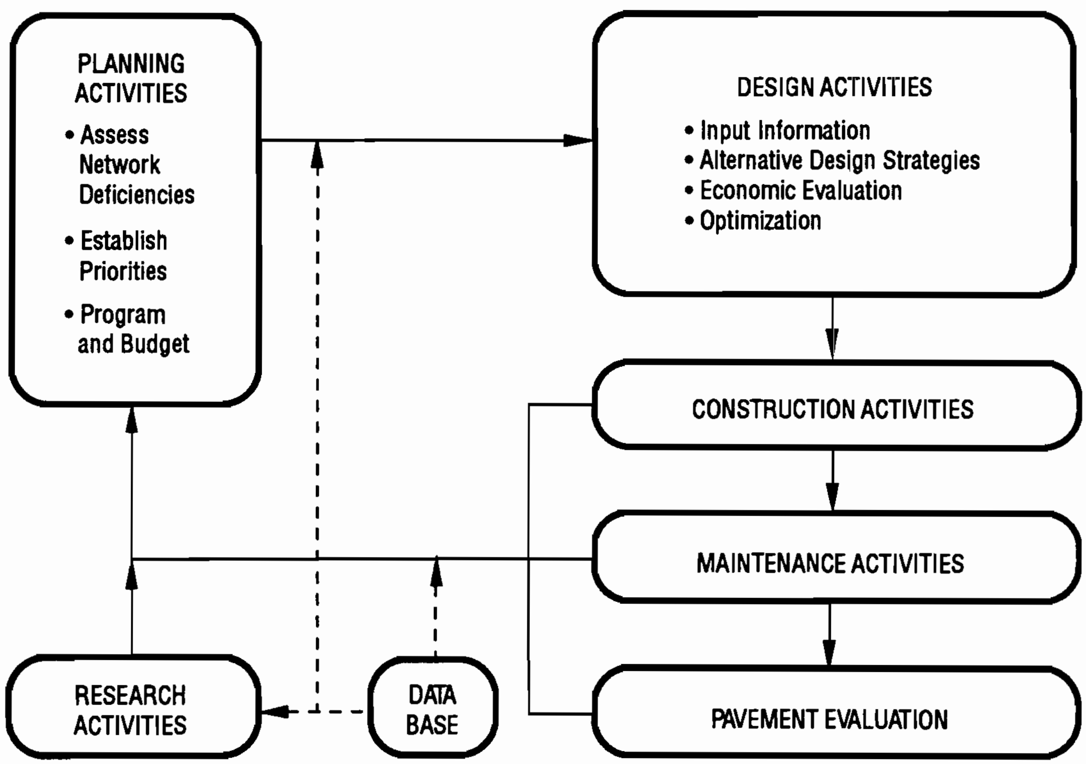


Fig 1.1. Basic components of a PMS (Ref 14).

savings. A good example is the AASHO Road Test performance prediction model (Ref 1), which is used worldwide.

It has been observed that, because of the excellent routine maintenance, the riding quality of Texas CRCP remains almost unchanged with time, even when a structure is approaching the end of its structural or economical life. Therefore, the AASHTO present serviceability index (PSI) (Refs 1 and 2) is not a reliable indicator of the performance of Texas CRCP, as shown in Fig 1.2. On the other hand, punchouts and patches were found to be good indicators of CRCP performance, as shown in Fig 1.3. These distress manifestations were used to calculate a distress index capable of capturing this important characteristic of the CRCP deterioration.

This distress index was termed the Zeta-index, or Z-score, and it is given by

$$Z = 1.0 - 0.0071 * MPO - 0.3978 * SPO - 0.4165 * PAT \qquad (1.1)$$

where

Z = distress index, or Zeta-score,
MPO = ln [(number of minor punchouts/mile) + 1],
SPO = ln [(number of severe punchouts/mile) + 1],
PAT = ln [(number of patches/mile) + 1], and
ln = natural logarithm.

In Eq 1.1, the variable PAT (number of patches) has the highest impact on the Z-score, followed by severe and minor punchouts. A perfect pavement has Z = 1.0. A CRC pavement becomes a candidate for rehabilitation when its Z-score reaches zero. The Z-score has already been used by the SDHPT for scheduling pavements for rehabilitation services.

However, the problem of an adequate model for performance prediction and design of Texas CRCP remained unsolved. The results reported in Research Report 472-2 (Ref 6), together with other experimental evidence, suggest that a good initial general format for a CRCP performance prediction model is

$$Z_i - Z_c = f\ (D, E_c, f_f, k, J, C_d, W_{eq}) \qquad (1.2)$$

where

Z_i = initial Z-index (distress free),
Z_c = current Z-index,
D = slab thickness,
E_c = elasticity modulus of the PC concrete,
f_f = flexural strength of the PC concrete,
k = modulus of reaction on top of subbase,
J = load transfer coefficient,
C_d = drainage coefficient, and
W_{eq} = cumulative equivalent single axle loads.

A design model capable of predicting the pavement life in terms of number of cumulative equivalent single axle loads is also desirable for design purposes. This model could have the following initial format:

$$W_{eq} = f\ (D, E_c, f_f, k, J, C_d, Z_i - Z_c) \qquad (1.3)$$

where all the variables are as in Eq 1.2, and the term $[Z_i - Z_c]$ would be used in the design model as the constant 1 to represent the Z-score variation experienced by a pavement that goes from a perfect condition (Z = 1) to the point of overlay (Z = 0).

OBJECTIVES

The ultimate objective of this research was to study CRCP performance in order to recommend long-term monitoring procedures and to develop suitable

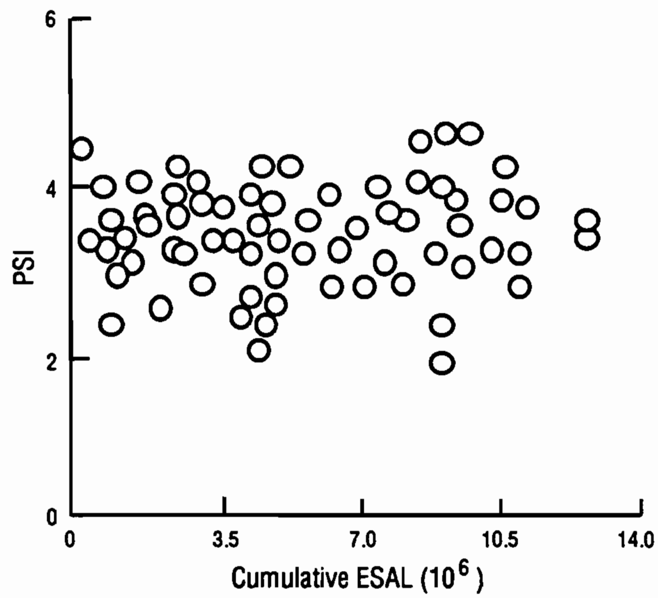


Fig 1.2. Inadequacy of PSI as indicator of Texas.

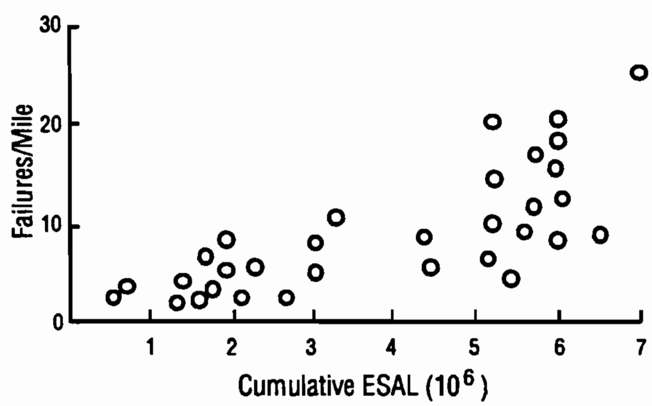


Fig 1.3. Adequacy of distress manifestations as indicators of Texas CRCP performance (Ref 8).

performance and design models for CRCP. This overall objectives were

(1) to determine the data needs for the models;
(2) to develop procedures for collecting the required data;
(3) to undertake the survey and to store the new data in the CRCP data base;
(4) to develop procedures for back-calculating material characterization parameters from the field data;
(5) to conduct a comprehensive review of the available data for the input variables for the model and to establish the model uses and limitations;
(6) to review the literature on statistics for selecting appropriate methods for calibrating pavement deterioration models;
(7) to use the results above, plus the previously existing historical condition survey data, in the model calibration process; and
(8) to suggest ways for implementing the model and the long-term monitoring procedures and to make recommendations for further improving the results of this study.

RESEARCH APPROACH

The approach adopted to attain the objectives listed above is based on a compromise between the ability of the sample to represent actual conditions and the accuracy of the data in the sample used for performance studies. The desired models and long term monitoring procedures must represent the entire state of Texas; therefore, data are required on a network-level basis. The feasibility of a network-level survey sampling over 7,000 lane-miles of pavement obviously requires expeditious data collection procedures. Such data usually present only a rough overall picture and lack the high accuracy desired for model calibration. However, the models and monitoring procedures under study are meant to be implemented on a statewide basis, and trading accuracy for quantity would decrease the the ability of the sample to represent actual conditions, thus producing results biased towards the conditions prevalent in the small sample. Since biased results are less desirable than results that, albeit lacking high accuracy, truly represent the entire network, it was decided to emphasize the ability of the sample to represent prevailing conditions.

For the organization of the research, the network-level data were classified as

(1) already available in the CRCP data base;
(2) necessary for the model calibration but completely unavailable and requiring development of field data collection procedures;
(3) necessary for long-term performance studies but completely unavailable and requiring development of field data collection procedures; or
(4) existent but unavailable on the CRCP data base and requiring retrieval and further implementation in the data base.

Type 1 data were primarily condition survey and pavement characteristics. They were used to develop the experimental factorial and the distress index mentioned above (Ref 6). Table 1.1 shows a summary of the available condition survey data.

Data types 2 and 3 comprised data for structural evaluation of the pavement, and they were termed

TABLE 1.1. SUMMARY OF THE AVAILABLE CONDITION SURVEY DATA

Distress Manifestation	Type	Intensity	Condition Survey Year					
			74	78	80	82	84	87
Cracking	Transverse	Minor	•	•				•
		Severe	•	•				•
	Longitudinal		•					
	Localized	Minor	•					
		Severe	•					
Spalling		Minor	•	•	•	•		
		Severe	•	•	•	•	•	
Pumping		Minor	•	•	•	•		
		Severe	•	•	•	•		
Punchout		Minor	•	•	•	•		•
		Severe	•	•	•	•	•	•
Patch		AC	•	•	•	•	•	•
		PCC	•	•	•	•	•	•
Crack Spacing	Transverse					•		
Reflected Cracks							•	
Overlay Bond Failure						•		

diagnostic data in this study. Data type 4 consisted basically of additional traffic data and overlay thickness data. The procedures for collecting and storing data types 2, 3, and 4 are discussed later in this report.

Once stored, the data were used to obtain material characterization parameters of the pavement, and finally to calibrate the model. The overall research approach of this study is depicted in Fig 1.4. In this figure, the boxes with a shaded background correspond to existent results, while the others correspond to phases of this study.

Scope and Organization

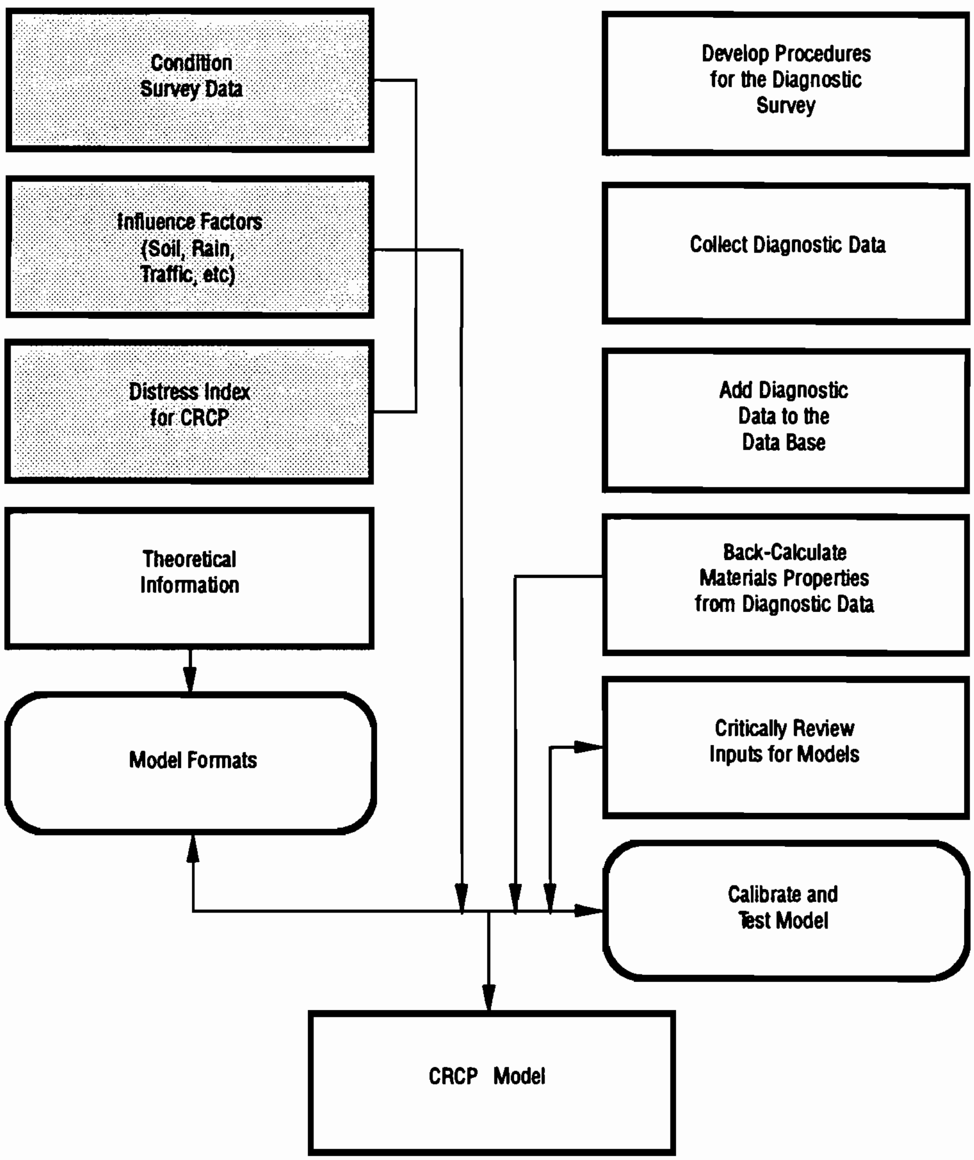
Chapter 1 gives background information, states and defines the objectives, and explains the basic approach used to meet those objectives.

Chapter 2 describes and discusses the procedures to obtain structural parameters of the pavement from the diagnostic data and gives the results obtained.

Chapter 3 discusses the possible impacts that their use as variables in a model can have on the final results and suggests alternatives for overcoming some of the limitations and inaccuracies inherent in the available data.

Chapter 4 documents the calibration process of the performance model and discusses the results.

Chapter 5 presents the major conclusions, suggests implementation procedures and practical uses for the model, and presents recommendations for further research in the area.



Note: Unshaded boxes indicate work pertaining to this study.
Shaded boxes indicate previous results used in this study.

Fig 1.4. Research approach.

CHAPTER 2. BACK-CALCULATION OF MODULUS OF REACTION AND ELASTIC MODULUS OF PC CONCRETE FROM DEFLECTION DATA

INTRODUCTION

Pavement performance studies require that the component materials be characterized. For rigid pavements, materials are characterized either by the elastic moduli of the layers or by the modulus of reaction on the top of the subbase in conjunction with the elastic modulus of the PC concrete. Since records of design values are frequently unavailable, the classical approach to obtaining elastic moduli is through back-calculation from deflection data. This back-calculation problem is generally tackled using some computer program capable of inverse application of layered theory. Several programs have been reported for inverse application of layered theory, and there seems to be no consensus as to which yields the best results. In the case of rigid pavements, there is a more fundamental question: which approach is better for back-calculation of materials properties, layered or plate theory? The back-calculation of material characterization parameters was used in this study, to compare the two different approaches. This chapter describes the two approaches and discusses the results obtained on a comparative basis.

NATURE OF DEFLECTION DATA USED IN THIS STUDY

INTRODUCTION

A significant part of this research was devoted to the development procedures for conducting a network-level survey on CRCP, which was intended to collect data for structural evaluation of the CRC pavements and was termed a diagnostic survey. The data collection procedures were developed considering long- and short-term research needs. The short-term needs relate basically to the inputs for the performance model, i.e., materials properties and load transfer coefficients. Already anticipated future uses of these data include non-destructive discontinuity detection, performance-oriented reinforcement design, study of edge and corner load conditions, and determination of the benefits of tied shoulders. Consequently, the following types of data were collected:

(1) deflection,
(2) crack width, and
(3) pavement temperature.

A brief description of the nature of the deflection data is presented in this chapter. Research Report 472-4 (Ref 47) documents the development of procedures for collecting these data, and Research Report 472-5 (Ref 46) presents a comprehensive description of the preparations for and the conducting of the diagnostic survey.

DESCRIPTION OF THE EQUIPMENT AND OF THE OPERATING PROCEDURE

The Texas State Department of Highways and Public Transportation (SDHPT) has several Falling Weight Deflectometer (FWD) units, and they were used for taking the deflection measurements used in this study. The FWD units were operated by SDHPT personnel in cooperation with the Center for Transportation Research (CTR).

The FWD is a trailer-mounted device that can be towed by any standard car or van at normal highway speeds. It applies an impulse load to the pavement by dropping a known mass from a pre-determined height. The mass falls on a foot plate connected to a rigid base plate by rubber buffers, which act as springs. The peak force acting on the surface where the mass falls, which is measured by a load cell, can theoretically be calculated using the following relationship:

$$P = \sqrt{2mghk} \tag{2.1}$$

where

P = peak force,
g = acceleration due to gravity,
h = height of drop of the mass,
m = mass of the FWD, and
k = spring constant.

The transient-pulse-generating device is the trailer-mounted frame, which is capable of causing a given preset mass configuration to fall from four different preset heights in a movement perpendicular to the surface. The assembly consists of the mass, frame, loading plates, and rubber buffer. Figure 2.1 depicts a scheme of the FWD with one of the possible geophone configurations.

The peak deflections are calculated by integrating the impact velocity, which is proportional to the output voltage in the velocity transducers, also termed geophones or sensors. The FWD can be viewed as a testing technique capable of obtaining deflections through measurements of a surrogate variable, the velocity.

The available FWD units can provide seven different deflection measurements per test. One of the geophones is located in the center of the loading plate, while the six remaining geophones are positioned along the raise/lower bar, up to 7 feet from the center of the loading plate, to the front and to the rear. The system includes a computer, which automatically records data from field testing and also accepts keyed-in information. The routine test procedure, from Research Report 387-1 (Ref 43), is as follows:

(1) Select and secure the mass configuration; this is usually done before travelling to the test site.
(2) Position the trailer on the pavement so that the marked test location is directly below the center of the loading plate.
(3) Turn on the processing equipment and the computer, which are carried in the towing vehicle.
(4) Program in the computer a drop height and a number of drops per test point. When the operator enters the "RUN" command, the FWD loading assembly is lowered to the pavement surface, the mass is dropped the programmed number of times from the pre-programmed height, and the assembly is raised again. This step typically lasts about two minutes.
(5) Inspect the data displayed on the computer screen. The operator can enter a "SKIP" command within a pre-programmed time if it is decided that the data should not be recorded; otherwise, the deflection data, the peak force magnitude and site identification information are stored and printed.

Due to the added difficulties in assembling, it was decided to use whichever SDHPT standard configuration was available at the particular FWD unit in use, and Step 1 was skipped.

PROCEDURE FOR COLLECTING THE DATA

The deflection measurements should provide data for estimating material characterization parameters and load transfer coefficients. They should also permit studies of corner and edge load conditions. In order to fulfill these needs, it is necessary to have the following deflection data from each test section:

(1) at the interior of the slab, to provide data for back-calculating material characterization parameters,
(2) at an edge, but far enough from a discontinuity to avoid its influence, to provide data for future studies of edge loading conditions,
(3) at both sides of a crack, to provide data for calculating load transfer coefficients, based on the idea that comparisons between deflections measured at a discontinuity and those taken in the interior of the slab are good indicators of load transfer (Ref 45), and
(4) at a corner, to provide data for future studies of corner load conditions.

Figure 2.2 shows the layout for making deflection measurements in an non-overlaid section, which is divided into five subsections, each consisting of a replicate of the layout of the test locations.

In the case of overlaid sections, load transfer studies are possible only if reflective cracks, which repeat the pattern from the pavement underneath, are visible. However, layer moduli can be back-calculated and edge

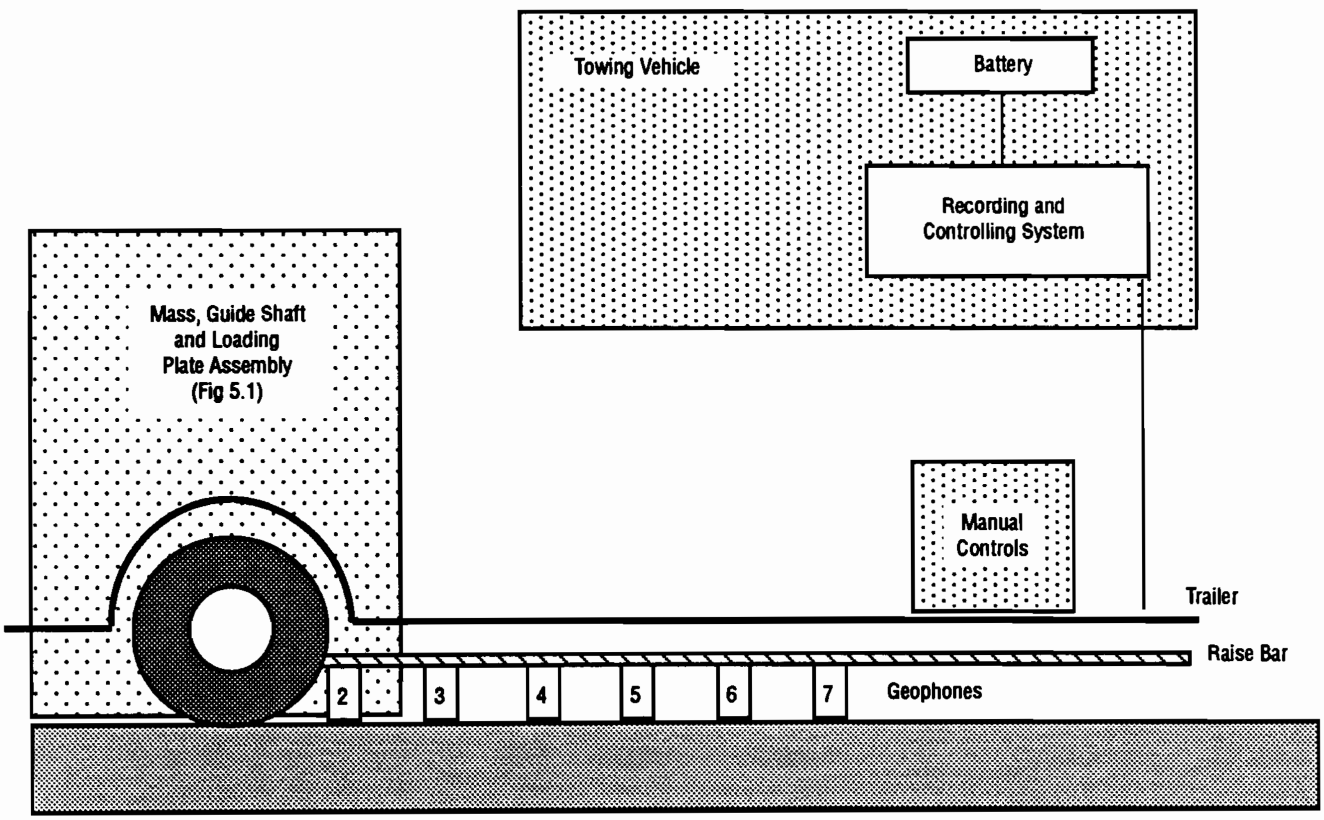


Fig 2.1. Falling Weight Deflectometer (Ref 43).

loading conditions can be examined if enough replicates are available to compensate for the high probability of selecting a test location over a discontinuity. Figure 2.3 shows the layout for deflection measurements in an overlaid section. The section is divided into ten replicates, or subsections.

The number of replicates per test section was decided based on the time available in the field. Since spending more than about one and a half hours per test section would not have been realistic, the maximum number of replicates was set at five for non-overlaid test sections and at ten for overlaid sections.

COLLECTION AND SUMMARY OF THE DEFLECTION DATA

During the diagnostic survey, five different FWD units that had either one of two standard geophone configurations already assembled were used. These configurations are shown in Figs 2.4 and 2.5. They were termed A and C, according to an existing convention of the Center for Transportation Research. The procedure for collecting deflection data was strictly followed by the field crews. Table 2.1 presents a summary of the deflection data taken at the interior of the slab. Distances between cracks around each measurement location were

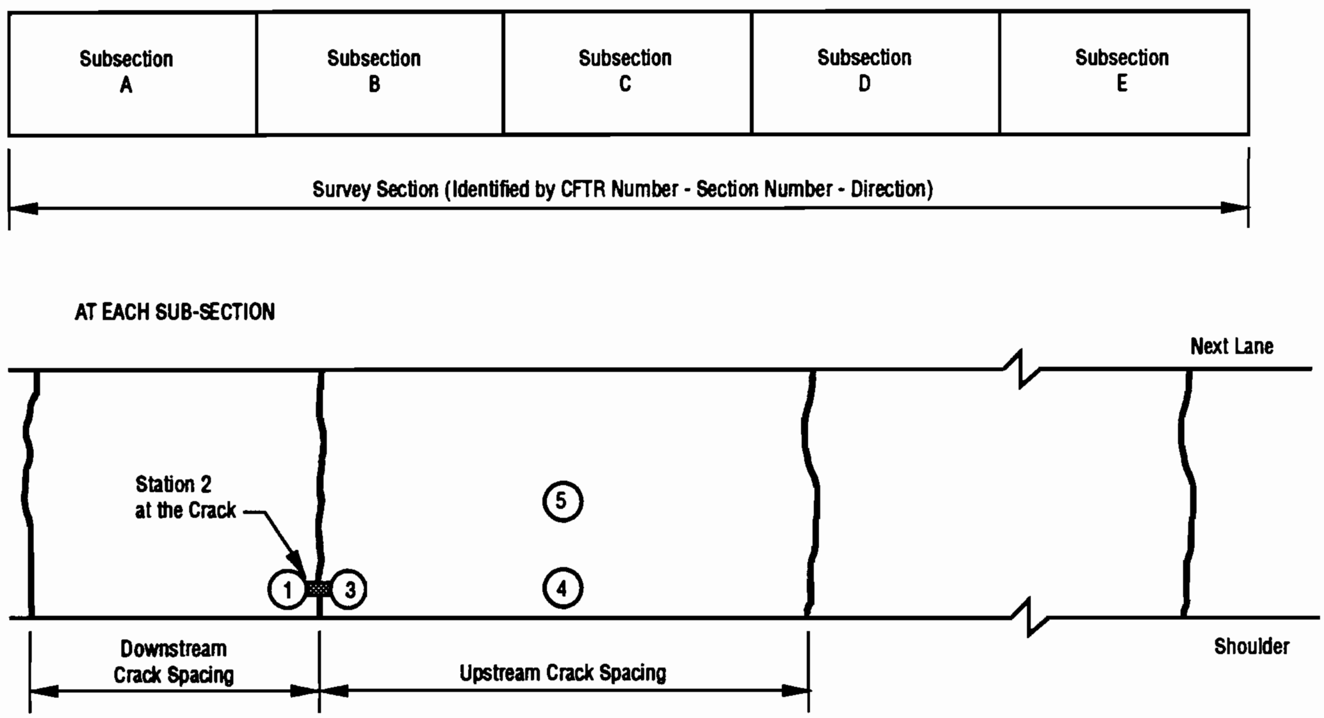


Fig 2.2. Layout for deflection measurements in a non-overlaid section.

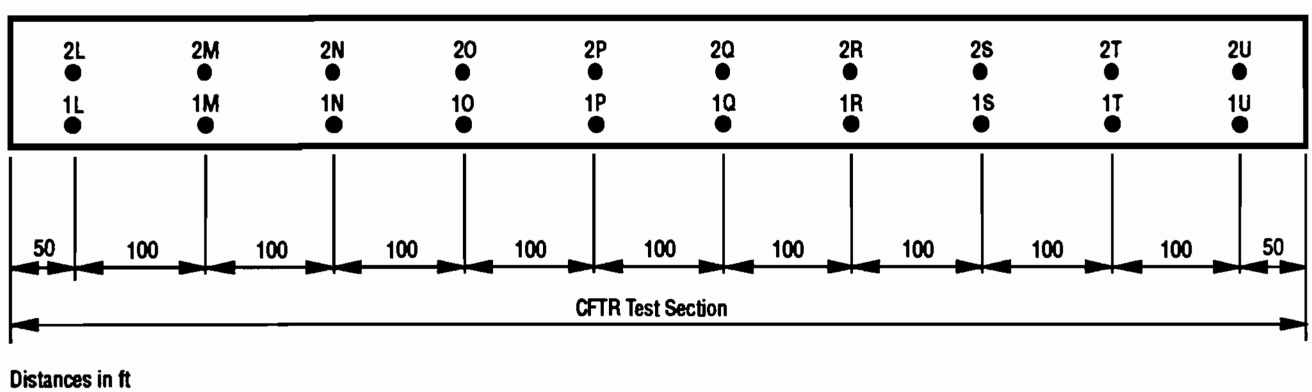


Fig 2.3. Layout for deflection measurements in an overlaid section.

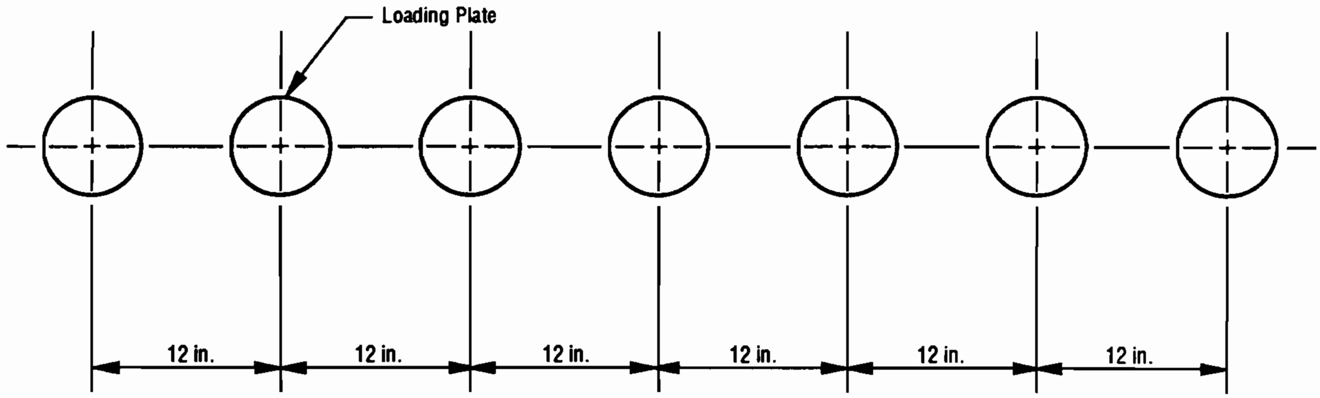


Fig 2.4. Geophone configuration A.

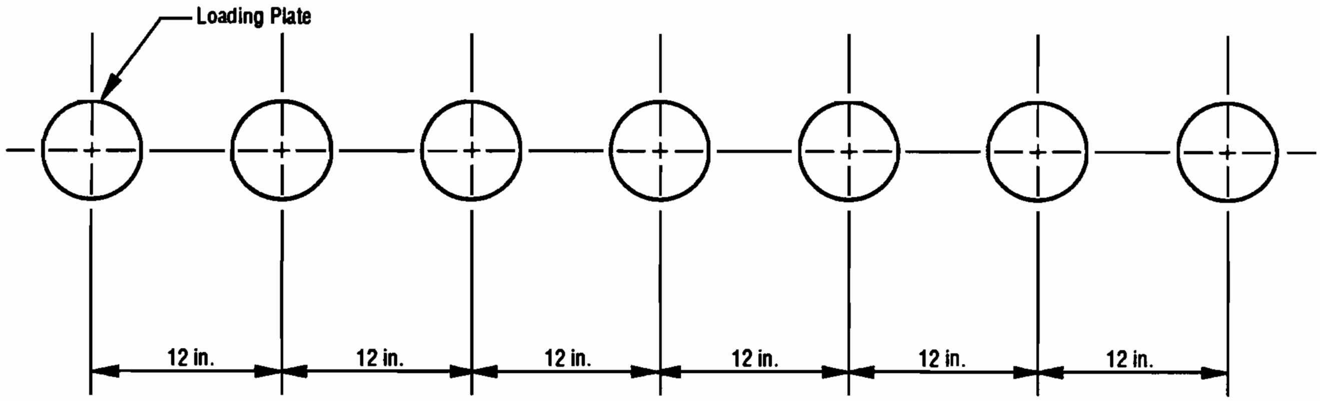


Fig 2.5. Geophone configuration C.

TABLE 2.1. SUMMARY OF THE DEFLECTION DATA

Geophone Configuration	Section	Variable	Number of Sections	Mean	Standard Deviation	Minimum Value	Maximum Value
A*	Overlaid	Load	41	16226	695	14656	18592
		Deflection		7.9	3.0	1.3	40.1
A*	Non-Overlaid	Load	27	17198	1065	15152	19064
		Deflection		6.5	2.7	2.2	14.5
C*	Overlaid	Load	66	17109	2085	9368	22560
		Deflection		10.0	4.5	3.7	34.8
C*	Non-Overlaid	Load	124	1653	397	13680	18232
		Deflection		6.8	2.1	1.7	18.7

*According to Figs 2.4 and 2.5.
Deflections measured under the load, in 0.001 in.

also recorded on a field form. Research Report 472-5 (Ref 46) describes the data collection in detail. Figure 2.6 depicts typical deflection basins for stations in a non-overlaid subsection, and Fig 2.7 shows typical basins in an overlaid subsection.

STORAGE AND REVIEW OF DEFLECTION DATA

After the survey, the raw FWD deflection basins were checked for departures from the expected pattern. At an early stage, plots of deflection basins for two entire Districts (5 and 13) were produced and physically inspected, in order to determine what to expect from field data. Once the typical departures were established, an SAS program was written to flag the basins presenting these departures, with an allowance of 5 percent for error. The automated check consists of a practical, fast and error-proof way of examining the deflection basins. It can be used as a part of any other SAS program that retrieves and/or analyzes the data whenever the user wants to make sure that only the appropriate basins are considered.

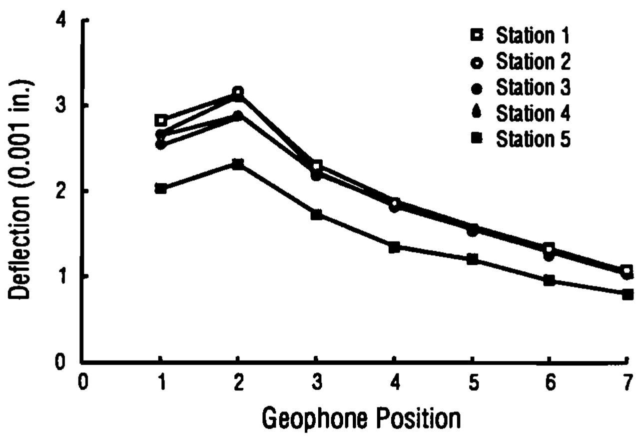


Fig 2.6. Deflection basins in a typical non-overlaid subsection.

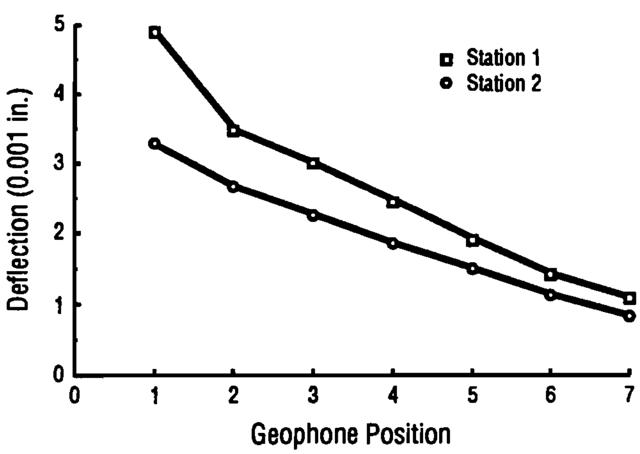


Fig 2.7. Deflection basins in a typical overlaid subsection.

Consequently, deletions in the data set were made only when it was evident that the reliability of that piece of data was questionable.

It was found that FWD results to be inconsistent for low drop heights (small loads). For CRCP, which is usually very stiff, it is likely that small loads are not enough to activate the sensors correctly. When the departure from the expected pattern was consistent for all four drop heights, it seems safe to conclude that some discontinuity was being detected. Districts 17 and 19 present good examples of this, which could be useful for studying discontinuity (i.e., void) detection with the FWD. In District 15, all geophone 4 readings were substituted for values linearly interpolated between geophones 3 and 5, because the output of the checking program indicated malfunctioning of geophone 4.

THEORETICAL BACKGROUND

The majority of solutions available for a pavement structure subject to load were developed for calculating deflections and stresses due to load in a pavement of known characteristics. For a rigid pavement, this problem can be solved using either layered theory or plate theory.

Both layered and plate theories solve the rigid pavement structure for effects of external load only. In fact, an important assumption of most pavement design methods is that effects of temperature or moisture are accounted for by stress relief devices, such as joints, so that the design process has to consider only the effects of load (Ref 49). Although this practice can be reasonable for design purposes, it may cause some problems for analysis of field deflections. Temperature and moisture differentials are always present in the field to some extent, and they may cause the measured deflections to be due to a combined effect of load, curling, and warping. A study carried out on an instrumented slab at Balcones Research Center showed that temperature and moisture differentials had significant effects on deflections for corner and edge load positions but not for the interior load position (Ref 28). Therefore, for the back-calculation procedure it can be assumed that all interior deflections are due to load only, and. thus the following approaches to the problem of back-calculating elastic and reaction moduli are possible for a non-overlaid section:

(1) use layered theory to obtain the elastic moduli of the layers and then use those to obtain the moduli of reaction on top of subbases or
(2) use plate theory to obtain directly the elastic moduli of the PC concrete and the reaction moduli on top of the subbases.

According to plate theory, Eq 2.2 describes the equilibrium of an elastic thin plate during bending. The constitutive equation derives from application of the generalized Hooke's law (Ref 18):

$$\frac{\partial^4 w}{\partial x^4} + \frac{2\partial^4 w}{\partial x^2 \partial y4} + \frac{\partial^4 w}{\partial y4} = \frac{12(1-\mu^2)}{ED^3}[p - kw] \quad (2.2)$$

where

p = load,
D = plate thickness,
w = deflection,
μ = Poisson's ratio of the plate,
E = elastic modulus of the plate,
k = modulus of reaction on top of the foundation, and
x, y = coordinates of the plane of the plate.

Equation 2.2 is based on the following additional assumptions about the plate structure:

(1) the middle plane remains neutral during bending;
(2) plane sections perpendicular to the middle plane remain plane during bending; and
(3) the vertical deflections are small compared to the plate dimensions.

Layered theory is a procedure originally conceived to calculate stresses and deflections in an axi-symmetric structure composed of n – 1 elastic layers resting on an elastic half-space (the n^{th} layer) and subject to a static load perpendicular to the top of the first layer. Table 2.2 compares the underlying assumptions of both theories.

Although the simplifications of the assumptions underlying both theories have already been exhaustively discussed (Refs 4, 14, 31, 32, 33, 44, 48, and 49), their impacts on back-calculation of parameters are not yet totally established.

The limitations due to the assumptions summarized in Table 2.2 will always affect any results obtained with the application of these theories. However, most back-calculation procedures based on iterative inverse application of layered theory have additional drawbacks that hold even in the case of a hypothetical pavement that meets the underlying assumptions of layered theory.

(1) The solution is not always unique, i.e., there may be more than one set of values of elastic moduli that will cause a certain pavement, when subject to the field load, to respond with the measured deflections. An example of non-uniqueness of moduli, obtained using program ELSYM5, can be seen in Fig 2.8.
(2) The method is very sensitive to the accuracy of the initial estimates of the moduli (seed moduli), because layered theory programs are based on complex iterative procedures that do not permit the difference between the calculated (Δ_c) and measured (Δ_m) deflections to be reduced to a mathematical function of the moduli. Therefore, instead of seeking the actual overall minimum point of the hypothetical function ($\Delta_c - \Delta_m$), the program searches around the seed moduli for a local minimum, which will be close to the desired overall minimum only if accurate initial estimates for the moduli are available. This situation is illustrated in Fig 2.9.
(3) The modulus of reaction on top of the subbase (k) is a parameter that represents the elastic stiffness of the springs in a Winkler foundation. It cannot be calculated directly from layered theory, because this theory considers all layers as elastic solids. Relationships between k and the elastic moduli of the layers have to be used to obtain k-values, and this practice introduces additional errors in the process, because it assumes a direct and unique relationship between parameters from two different structural models.
(4) For CRC pavements, the assumption of axi-symmetric layers is rigorously met only in the ideal case of 100 percent load transfer across the cracks. Figure 2.10 shows a comparison between the observed crack spacings in the test sections and the FWD sensor positions. Since the maximum observed crack spacing at the measurement stations was 12 feet, the best situation is when the farthest sensor is

TABLE 2.2. LAYERED AND PLATE THEORY ASSUMPTIONS COMPARED

Assumption/Feature	Plate Theory	Layered Theory
Number of layers	Two: plate and foundation	At least five, depending on the program
Layers solved for	Plate only	All
Boundary conditions	Bottom layer homogeneous	May consider presence of rigid deep bottom layer
Load positions with respect to plate edges	Interior, at an edge or at corner	Structure is axi-symmetric with respect to load
Characteristics of layers	Upper layer = elastic solid Lower layer = dense liquid	All layers = elastic solids
State of stress considered	Only horizontal, i.e., stresses due to pure bending	Vertical, radial, and shear

over a crack, and, in most cases, only the sensor under the load can be safely assumed to be far from discontinuities.

Back-calculation through closed-form solutions from plate theory has the advantage of overcoming drawbacks 2, 3 and 4 above, because the elastic modulus of the PC concrete and the reaction modulus can be calculated directly from a system of equations that have the desired parameters as unknowns. In addition, since only two sensors are needed to derive a system of two equations, use of the sensor under the loading plate and its nearest neighbor guarantees more distance from discontinuities, and this ensures better validity of the assumption of a continuous plate, for a considerable number of test sections. This choice of sensors has the additional advantage of drastically decreasing the problem of non-uniqueness of solution. In the case of plate theory, non-uniqueness of solution is caused by deriving systems of two equations using more than two deflection measurements.

If geophone configuration C in Fig 2.5 is used, the strategy to ensure adherence to the assumption of a continuous plate automatically ensures the uniqueness of the solution. Where configuration A is used (Fig 2.4), either the sensor located one foot to the front or the sensor located one foot to the rear of the loading plate can be chosen for deriving the equations, in addition to the sensor under the loading plate. Since the mean deflections in these two sensors are 5.8 and 5.7, the standard deviations are 1.9 and 2.0, and, since no significant difference exists between their deflections, uniqueness of the solution is guaranteed for practical purposes.

The contrast between the apparent advantages of the plate theory approach over the layered theory approach and the prevalence in the literature of procedures based on layered theory were the main reasons for this comparative study.

BACK-CALCULATION OF MODULI THROUGH LAYERED THEORY

BASIC APPROACH

The solutions of a pavement structure using layered theory depend on cumbersome numerical methods and a number of computer programs are available. In general, the inputs for this type of programs are:

(1) load,
(2) thickness of the layers,
(3) elastic and Poisson's moduli of the layers,
(4) coordinates of the positions where the output is desired, and
(5) eventually, other data, such as boundary conditions at interfaces and/or at the bottom layer.

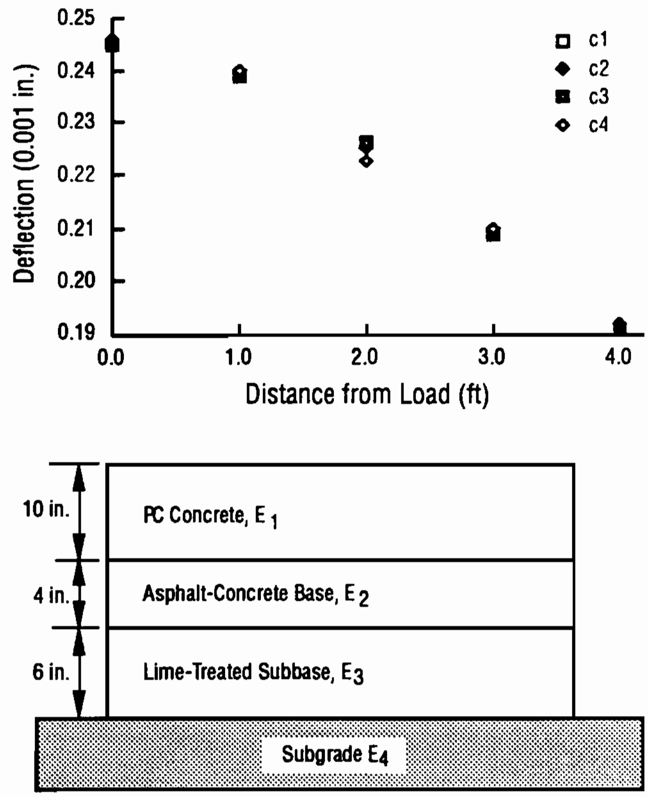


Fig 2.8. Example of non-uniqueness of back-calculated moduli (Ref 43).

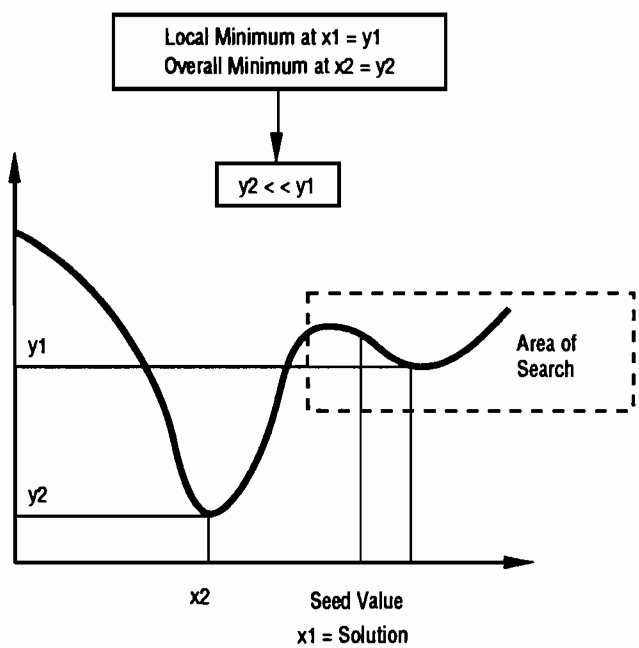


Fig 2.9. Conceptual difference between local and actual minima.

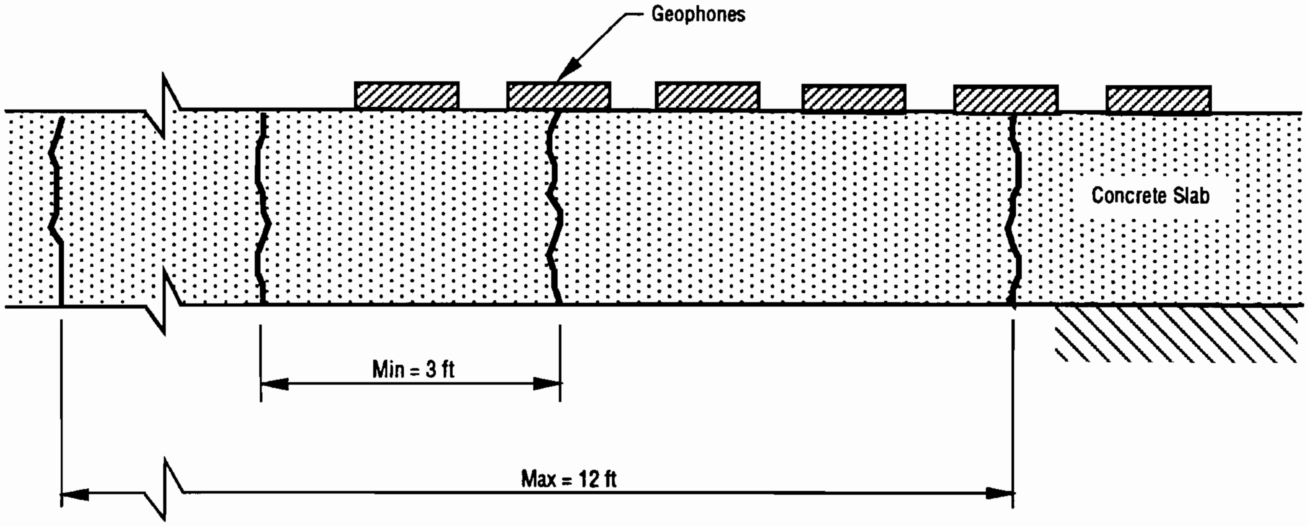


Fig 2.10. Geophone positions with respect to observed crack spacing.

The output consists of deflections and stresses due to the load, at the locations specified in the input. For this particular research need, the output should be the elastic moduli of the layers. There are a number of programs that can take deflection basins, geometric characteristics, and load as inputs, and, using some layered theory program, interactively select a combination of elastic moduli that results in a calculated deflection basin close to the measured one, for the same load (Refs 21, 27, and 43). Programs RPEDD1 and FPEDD1 (Ref 43), developed and available at The University of Texas at Austin, were used in this study. They are discussed below.

Once the elastic moduli are obtained, it is still necessary to obtain the moduli of reaction on top of the subbases (k). They were calculated using an equation relating subgrade modulus, subbase modulus, and subbase thickness to k. This equation, from Ref 2, is

$$\begin{aligned}\ln(k) = &-2.807 + 0.1253\,[\ln(D_{SB})]^2 + 1.062\ln(E_{SG}) \\ &+ 0.1282\ln(D_{SB})\ln(E_{SB}) - 0.4114\ln(D_{SB}) \\ &- 0.0581\ln(E_{SB}) - 0.1317\ln(D_{SB})\ln(E_{SG})\end{aligned} \tag{2.3}$$

where

- k = modulus of reaction on top of subbase,
- ln = natural logarithm,
- D = thickness,
- E = elastic modulus,
- SB = subscript for subbase, and
- SG = subscript for subgrade.

The derivation of this equation is explained in the *AASHTO Guide for Pavement Structures* (Ref 2), and it relies on the following basic assumptions:

(1) k can be calculated as

$$k = P / V \tag{2.4}$$

where

- P = total load (or force) applied to a 30-inch rigid plate resting on the top of the subbase and
- V = volume of soil directly beneath the plate that is displaced by the load (P).

(2) The volume (V) is calculated using layered theory, and this result is used to obtain k with Eq 2.4. Next, Eq 2.3 is derived using the k-values and the elasticity moduli and the subbase thickness used in the layered theory program that calculated the displaced soil volumes.

The approach above assumes a direct equivalency between the response to load of a series of unconnected vertical springs and the response to load of a homogeneous and isotropic elastic solid. This introduces errors in the back-calculated values, which will add to the errors caused by the limitations inherent in the layered theory.

PROCEDURE TO BACK-CALCULATE ELASTIC AND REACTION MODULI WITH LAYERED THEORY

Programs RPEDD1 and FPEDD1 (Ref 43), which stand respectively for Rigid and Flexible Pavement Evaluation based on Dynamic Deflections, were used in this study. They have several subroutines, of which the following were used in the back-calculation process:

(1) a self-iterative procedure to estimate in situ elastic moduli by fitting a deflection basin calculated through inverse application of the layered elastic theory program ELSYM5 to the measured basin and

(2) a procedure for correcting the temperature sensitive asphaltic-concrete modulus to the design temperature.

The first routine works by calling ELSYM5 to calculate the deflection basin due to the measured load, the known pavement characteristics, and the initial, estimated moduli of the layers (seed moduli). The measured deflection basin is compared to the calculated one, and, if the difference is greater than a preset acceptable value, new seed moduli are calculated, and the process continues either until the difference between calculated and measured deflections is acceptable or until the maximum number of iterations is reached.

Asphaltic-concrete moduli derived from the procedure described above represent the in situ values at the temperature of the test. For design and evaluation purposes, moduli at the design temperature are used. The second routine corrects these moduli by applying a correction factor, which consists of the ratio between moduli at design temperature and at test temperature. If laboratory correction factors are available they can be input by the user; otherwise, the program uses default correction factors from Ref 22, which was the case in this particular study.

The two programs used in this study are subject to the limitations of the layered theory approach. In order to obtain good initial estimates for the seed moduli, the following strategy was adopted:

(1) Use Ref 19 data for ranges of 28-day elastic moduli of PC concretes made with typical Texas aggregates to place tight limits on the modulus of the first layer. Data on pavement age, available in the data base, and typical elastic modulus growth with time (Ref 30) were used to extrapolate the 28-day elastic modulus to the modulus at the date of the survey.

(2) Data on ranges of elastic moduli of typical base layers in Texas (Ref 43) were also used as boundaries for the elastic modulus of the subbase.

This strategy was used to narrow the problem down in order to solve for the subgrade modulus, thus giving unique, reliable solutions. A set of computer programs was written to select the best deflection basin out of the available replicates in each test section, retrieve the rest of the appropriate data from the CRCP data base, apply the above strategy, and write the input files for RPEDD1 and FPEDD1. Appendix A shows a printout of these programs for FPEDD1. The programs for RPEDD1 are analogous.

The following steps were used to select a deflection basin for a test section.

(1) Select only the deflections corresponding to the maximum drop height, because lower load levels do not give reliable deflection measurements on CRCP (Refs 10 and 46).

(2) Select only the deflection basins corresponding to the interior load position, in order to avoid influence of temperature and moisture differentials (Ref 28).

(3) Eliminate deflection basins that depart from the expected pattern by more than 5 percent.

(4) When applicable, eliminate a previous deflection basin already submitted to RPEDD1 or FPEDD1 that yielded an error greater than 35 percent in the fitted basin.

(5) Calculate the normalized deflections for the remaining deflection basins.

(6) Using the normalized deflections, calculate the area under the deflection basin and select, as representative of a test section, the load and the basin corresponding to the median normalized area.

Program ELSYM5, called by RPEDD1 and FPEDD1, can consider the presence of a very stiff layer somewhere underneath the subgrade. If this layer exists at a depth less than about 20 feet, it has positive impact on the foundation support. No data were available on the presence or absence of a rigid layer, and information was sought from engineers with many years of experience with Texas highways. As a result, it was decided to assume that the rigid layers, when present, are too deep to have any influence. Whenever this assumption does not hold, the elastic moduli are overestimated, in order to account for the actual stiffening effect of the deep rigid layer. An evaluation was made of the amount of overestimation, through comparisons of results from two assumptions: no rigid layer and a 3-foot-deep rigid layer. The observed decreases in moduli remained within 20 to 30 percent.

PRESENTATION AND DISCUSSION OF LAYERED THEORY RESULTS

The results obtained with the procedure described above are shown in Appendix B. They consist of elastic moduli of the layers, the errors from RPEDD1 or FPEDD1, and the moduli of reaction on top of subbase, calculated using Eq 2.3. The frequencies of the overlay moduli are depicted in Fig 2.11. More than 50 percent of the values are greater than 400,000 psi, which is a reasonable result for good asphaltic concretes. Figures 2.12, 2.13, and 2.14 depict the results of PC concrete, subbase, and subgrade moduli, respectively. The PC concrete moduli show a tendency to cluster in three groups: 5.0, 5.5, and 5.8 million psi, reflecting the age groups of the test sections in the data base. The subbase moduli also show a tendency to cluster in two groups, 100,000 and 200,000 psi, because 80 percent of the test sections have either cement-treated or asphalt-treated subbases. The subgrade moduli do not show any tendency to cluster into groups, but their histogram is skewed. Figure 2.15 shows the back-calculated k-values. The maximum value is

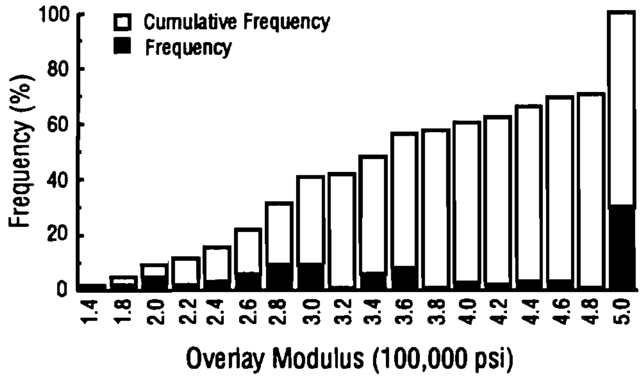


Fig 2.11. Frequencies of overlay moduli from layered theory.

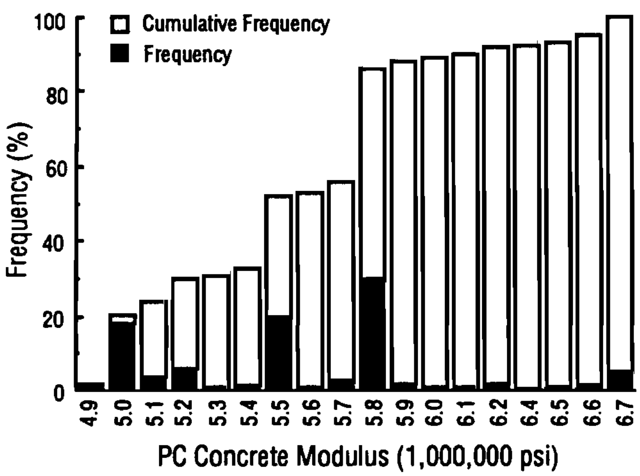


Fig 2.12. Frequencies of PC concrete moduli from layered theory.

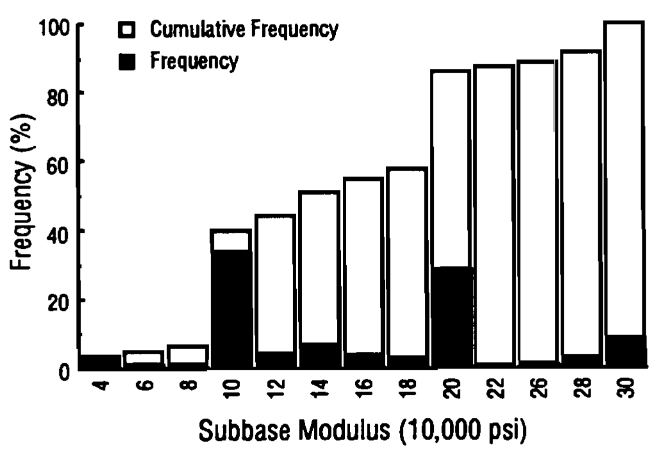


Fig 2.13. Frequencies of subbase moduli from layered theory.

3,608 pci, the minimum is 577 pci, and the mean is 1,721 pci. The magnitudes of these values are higher than the figures usually found in the literature (e.g., in Refs 1 and 2).

Although Eq 2.3 is a possible source of error that cannot be overlooked, the back-calculated elastic moduli do not always give good matches between measured and calculated deflections. The mismatches are caused by the errors due to drawbacks of the layered theory approach, such as the need for very accurate initial estimates of the moduli. Figure 2.16 shows the maximum percent of observed differences between calculated and measured deflections at each deflection basin, and an undesirably high frequency of deviations above 20 percent can be seen.

In summary, the magnitudes of k-values are higher than usually found in the literature, and the results cannot be considered accurate, because there are too many high deviations between measured and calculated deflections. The causes of this can be summarized as follows.

(1) Failures in assumptions of layered theory, especially the assumption of axi-symmetric layers (Fig 2.10), introduce errors in the results.

(2) Accurate information about the presence and depth of rigid layers underneath the subgrade was substituted for the assumption of the absence of a rigid layer, suggested by engineers with many years of experience on Texas highways. The moduli are overestimated when this assumption does not hold. The high k-values obtained with this method would indicate the error of this assumption if no other errors were affecting the results.

(3) The non-uniqueness of solution, in conjunction with the fact that accurate solutions depend on accurate estimates of seed moduli, can introduce significant errors.

(4) Estimation of k using relationships with elastic moduli of slab foundation layers is an additional source of errors.

Since what is actually needed for calibrating the model are moduli of reaction on top of the subbase and elastic moduli of PC concrete, closed-form solutions of the plate theory were sought in order to directly find k on top of the subbase, for the hypothesis that all deflection is due to load alone. This approach overcomes some of the drawbacks of the layered theory, but it can be used only for the non-overlaid sections.

BACK-CALCULATION OF MODULI OF REACTION ON TOP OF SUBBASE THROUGH PLATE THEORY

BACKGROUND

The most important closed-form solutions for the problem of a rigid pavement resting on a subgrade are derived from Westergaard (Ref 48). He solved an idealized structure consisting of a homogeneous, isotropic, and elastic plate resting on a foundation capable only of vertical reactions, which are assumed proportional to the deflection. The constant of proportionality was termed by Westergaard as the modulus of subgrade reaction. Today, it is frequently termed modulus of reaction on top of the subbase, to account for the presence of this layer.

Westergaard (Ref 48) solved the problem for a load uniformly distributed over an ellipse of semi-axes a and b, for three load positions: in the interior of the panel, at an edge, and at a corner. An important concept introduced by Westergaard is the radius of relative stiffness (L), defined as shown in Eq 2.5. The radius of relative stiffness accounts for the fact that the response to load of a panel resting on a foundation depends not only on the characteristics of the panel (E_c, D, and μ) but also on the strength of the foundation it is resting on, expressed by k. Thus

$$L^4 = (E_c D^3) / (12 (1 - \mu^2) k) \qquad (2.5)$$

where

- D = slab thickness,
- E_c = elastic modulus of PC concrete,
- μ = Poisson's ratio of PC concrete, and
- k = modulus of subgrade reaction.

For the interior load condition, Westergaard's solution gives the deflection at a point (x,y) within or near the loaded area (Eq 2.6). This load position is useful for avoiding effects of temperature and moisture differentials on deflections:

$$w = \frac{p}{8kL^2}\left[1 - \frac{a^2 + b^2 + 4x^2 + 4y^2}{16\pi L^2}\ \ln \frac{E_c D^3}{k\left(\frac{a = b}{2}\right)4} - \frac{a^2 + 4ab^2 + b^2}{16\pi L^2} + \frac{(a - b)(x^2 - y^2)}{2\pi L^2 (a + b)}\right] \qquad (2.6)$$

where

- w = deflection at point (x, y),
- ln = natural logarithm,
- x, y = coordinates where the deflection is calculated,
- P = load,
- L = radius of relative stiffness,
- k = modulus of subgrade reaction,
- a, b = semi-axes of the ellipse,
- D = slab thickness, and
- E_c = elastic modulus of the PC concrete.

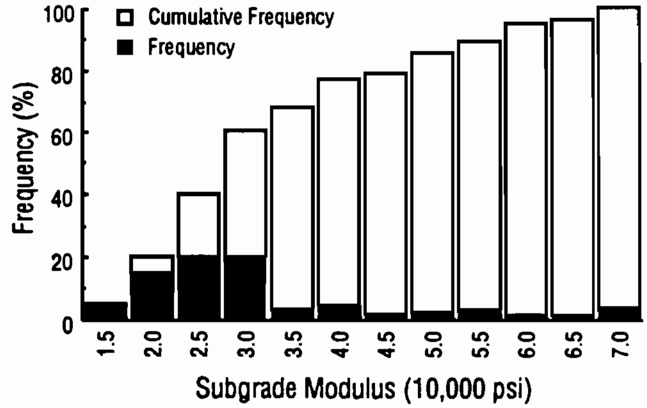


Fig 2.14. Frequencies of subgrade moduli from layered theory.

BACK-CALCULATION PROCEDURE AND RESULTS

Equation 2.6 relates deflections to pavement characteristics and modulus of subgrade reaction. For the CRCP test sections, w, P, D, a, and b are available. The coordinates x and y correspond to the sensor positions. The unknowns are E_c, k, and μ. However, since the range of variation of μ among different types of PC concretes is small and has little impact on the pavement response, it can be assumed equal to 0.15. The unknowns are the PC concrete elastic modulus (E_c) and the modulus of reaction on top of the subbase (k). This would require solving for a system of two equations derived from Westergaard's interior load equation (Eq 2.6) for k and E_c, for deflections at two different sensor positions. However, since the E_c values from layered theory were based on initial estimates using laboratory data, it was decided to substitute these E_c values in Westergaard's equation. This approach reduces the problem of solving one equation for k, and this equation can be obtained using the deflection under the loading plate, thus ensuring the best possible adherence to the assumption of continuity of the plate (see the loading plate position with respect to cracks in Fig 2.10).

Equation 2.6 is a monotonically decreasing function of k. Using straightforward algebraic manipulations, it can be stated as follows:

$$w - \left[\frac{A_1}{\sqrt{k}}\left[1 - \left(A_2\sqrt{k}\right)\ln\frac{A_3}{k} - A_4\sqrt{k}\right]\right] = 0 \qquad (2.7)$$

where w and k are as defined in Eq 2.6, and A_1, A_2, A_3, and A_4 are given by Eqs 2.8, 2.9, 2.10, and 2.11, respectively:

$$A_1 = \frac{P}{2.336\sqrt{E_c D^3}} \qquad (2.8)$$

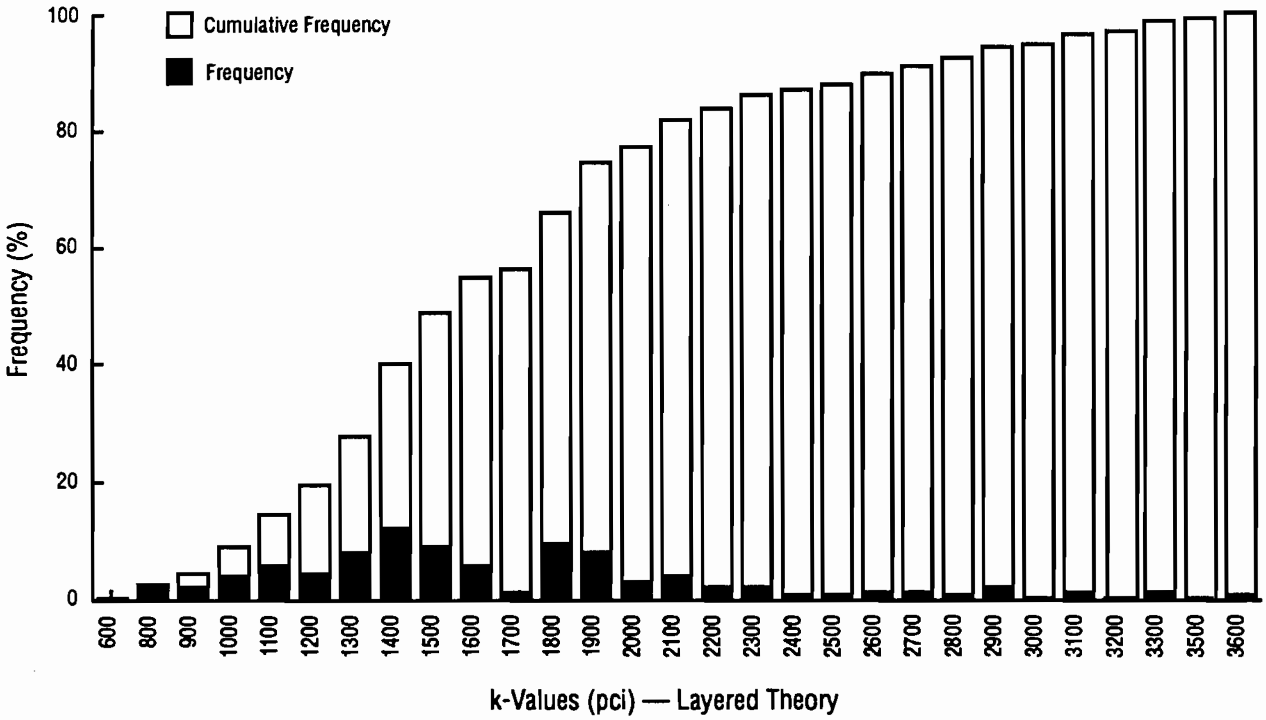


Fig 2.15. Frequencies of k-values from layered theory.

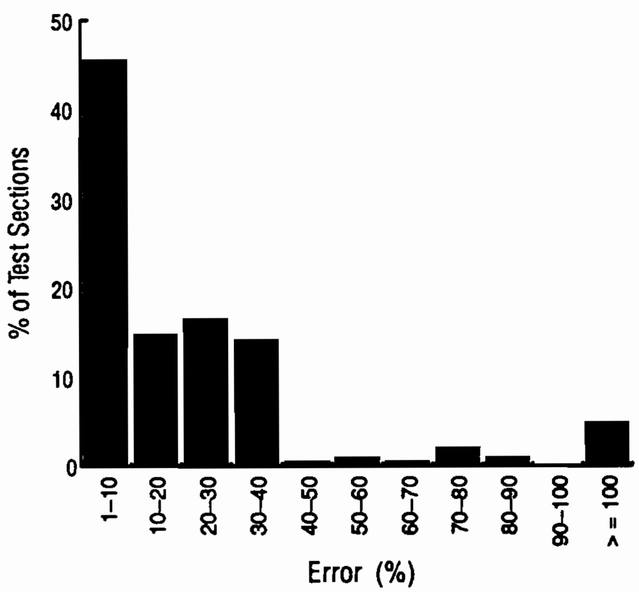


Fig 2.16. Percent differences between measured and calculated deflections.

$$A_2 = \frac{4.774}{\sqrt{E_c D^3}} \tag{2.9}$$

$$A_3 = \frac{E_c D^3}{1211.736} \tag{2.10}$$

$$A_4 = \frac{14.231}{\sqrt{E_c D^3}} \tag{2.11}$$

In Eqs 2.8 through 2.11, all parameters are as in Eq 2.6, and the Poisson's ratio of the PC concrete was assumed to be 0.15.

An efficient algorithm for solving Eq 2.7 must stop either when the difference between the deflection for the current k and the measured deflection is negligible or when the difference between two successive estimates of k is negligible. In addition, it must be possible to start from an initial interval for the root, rather than an initial estimate of it. The algorithm that satisfies these criteria is called the Bisection Method (Ref 7), and it is suitable for monotonic functions. The method consists of searching the root at increasingly smaller intervals, until either one of the convergence criteria is met. Figure 2.17 shows the method. The tolerance values were set at 10 pci for differences in k and at 10^{-10} for differences in deflections. Thresholds for k in pci were set in the interval between 20 and 2500. All calculated k-values were within this interval, except the one that corresponds to the 2,500 pci value.

The input file for the bisection method program was written by a SAS program that selects an appropriate deflection basin for each test section, applying the same criteria used for the layered theory; retrieves from the data base the rest of the inputs; and calculates the parameters A_1, A_2, A_3, and A_4 (Eqs 2.8 through 2.11). For the sake of brevity, these programs are not reproduced here. The program that applies the bisection method is straightforward, and the program that writes its input file is similar in its important parts to the one depicted in Appendix A.

Appendix B presents the results of the moduli of reactions obtained through plate theory. Cumulative and simple frequencies of the obtained k-values can be seen in Fig 2.18. The minimum value was 139 pci, the maximum was 2,500 pci, and the mean was 463 pci. These magnitudes are in good agreement with values found in the literature (e.g., Refs 1 and 2).

COMPARATIVE ANALYSIS OF THE RESULTS

Figure 2.19 shows a direct comparison between the results from the two approaches. The k-values from plate theory are consistently about 1/3 to 1/6 of those obtained with layered theory. Both results are consistent, i.e., high/low k-values correspond for both approaches. The discrepancy between results obtained with the approaches is due to the differences in boundary conditions and assumptions about the structure response. For example, the programs used for inverse application of layered theory call ELSYM5 with the full interface friction option, whereas Westergaard's assumption about the plate foundation implies no load transfer due to plate/foundation friction. In addition, failures in assumptions that affect both approaches may affect the back-calculated k-values in different directions.

Summary plots of the k-values versus subbase and subgrade moduli from layered theory are depicted in Figs 2.20 and 2.21. No consistent trend of k with subbase modulus is apparent, but some growth of k with subgrade modulus can be observed.

Previous studies of CRCP performance (Ref 6) detected some influence of the interaction between potential subgrade swell and average annual rainfall in the number of distress manifestations observed in the CRCP test sections used in this study. Consequently, the consistency of the results can be verified by checking the significance of the influence of this interaction on the k-values.

The most powerful statistical procedure for testing the significance of external factors in a random variable is called analysis of variance (ANOVA). Statistical books, such as Costa Neto's (Ref 8), explain this procedure in detail and discuss the assumptions about the behavior of the random variable that are required for its validity. Since these assumptions are not true for the back-calculated k-values, a non-parametric procedure that permits the influence of external factors on the k-values to be detected was sought in the literature. In this case, the nearest neighbor – "discriminant analysis" is appropriate. Descriminant analysis is a non-parametric method for classifying observations into groups, using the nearest neighbor rule, which consists of sorting the

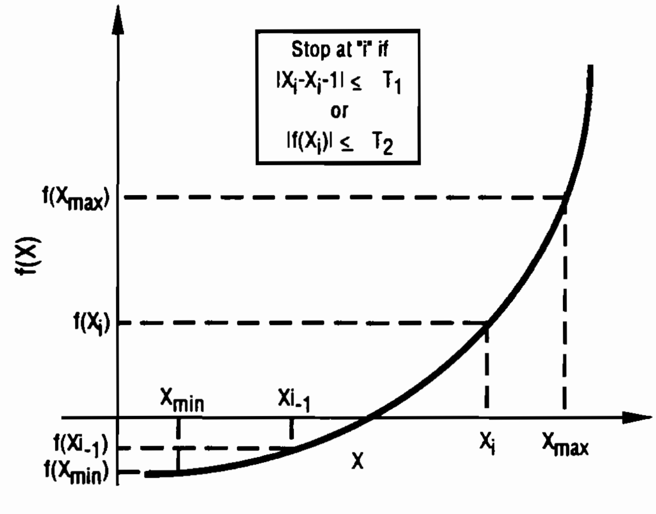


Fig 2.17. Bisection method.

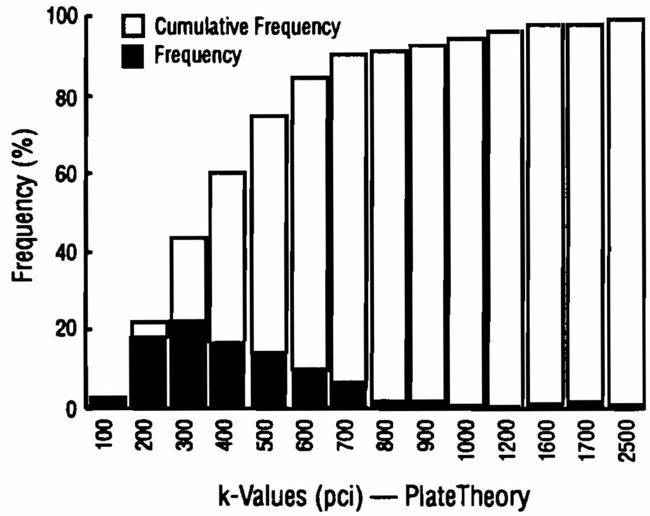


Fig 2.18. Summary of results from plate theory.

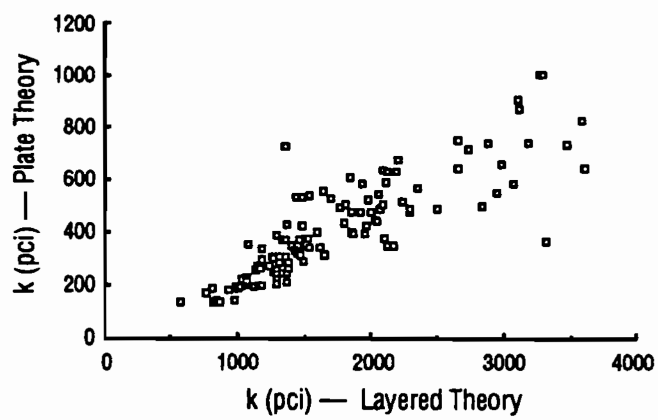


Fig 2.19. Comparison of results from layered and plate theories.

sample, looking at the n nearest neighbors of an observation, and classifying the sample into the group that contains the highest proportion of its n nearest neighbors.

Obviously, this type of test is less powerful than ANOVA. Non-parametric tests are always less powerful than an equivalent parametric test with assumptions met (Refs 5 and 8), and the nearest neighbor discriminant analysis cannot simultaneously take into account all possible combinations of levels of variables of interest, as ANOVA does. In addition, the only way to analyze interactions is to create artificial groups that represent the most important interactions and analyze only those cases.

The four quartiles of the average annual rainfall observed at the test sections were combined with qualitative data on the potential subgrade swell to construct the following groups:

(1) H1 = high subgrade swelling potential and average annual rainfall less than 28.4 inches.

(2) H2 = high subgrade swelling potential and average annual rainfall between 28.4 and 33 inches.

(3) H3 = high subgrade swelling potential and average annual rainfall between 33 and 38.4 inches.

(4) H4 = high subgrade swelling potential and average annual rainfall greater than 38.4 inches.

(5) L1 = low subgrade swelling potential and average annual rainfall less than 28.4 inches.

(6) L2 = low subgrade swelling potential and average annual rainfall between 28.4 and 33 inches.

(7) L3 = low subgrade swelling potential and average annual rainfall between 33 and 38.4 inches.

(8) L4 = low subgrade swelling potential and average annual rainfall greater than 38.4 inches.

The fact that a statistical method for classifying observations into groups can successfully identify the swelling potential and the amount of rainfall of a test section on the basis of its k-value is an indication that the k-values are influenced by the interaction between rainfall and potential subgrade swell. The results of classifications using layered theory k-values are depicted in Table 2.3, and the results using plate theory k-values are shown in Table 2.4.

For layered theory, only group L1 (low swell potential and average annual rainfall less than 28.4 inches) presents a significant number (75.7 percent) of correct classifications. This number is boldfaced in Table 2.3. It is concluded that k-values obtained through layered theory are not significantly affected by interaction between subgrade swelling potential and rainfall.

For plate theory, groups H4 and L1 presented more than 80 percent of the sections correctly classified. These numbers are boldfaced in Table 2.4. Since these groups represent the two extremes of the interaction under analysis, it is concluded that the interaction between rainfall and potential subgrade swell effects the plate theory k-values, but it is detectable only in extreme cases,

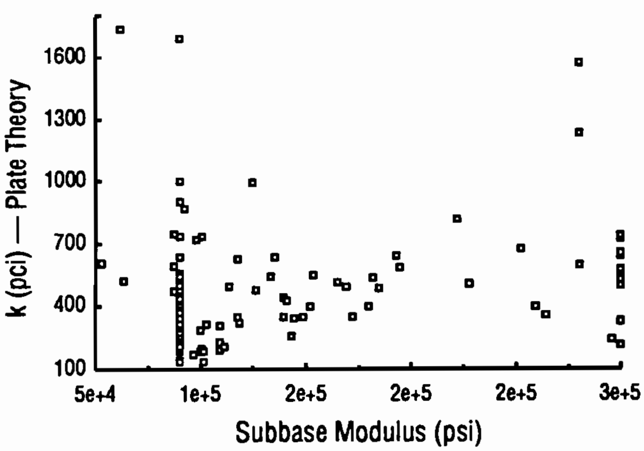


Fig 2.20. Plate theory reaction modulus versus subbase modulus.

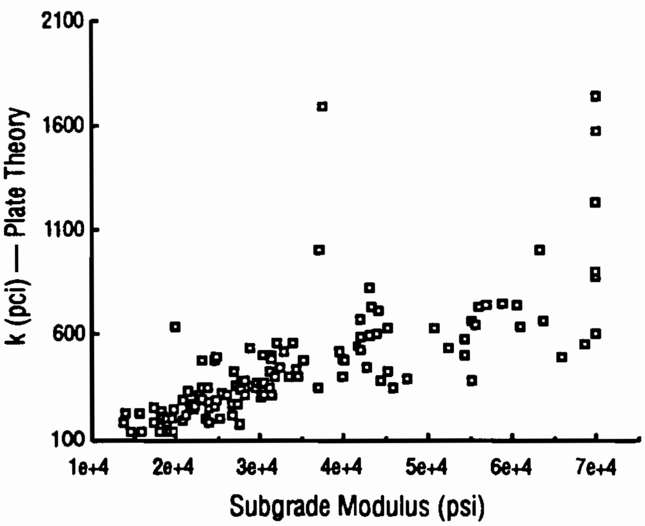


Fig 2.21. Plate theory reaction modulus versus subgrade modulus.

i.e., high swelling soils in the highest rainfall areas and low swelling soils in the driest areas. This finding indicates a better reliability of the plate theory approach. However, a more definitive conclusion requires further study with measured moduli for comparison, because there are a number of explanations for the independence of k-values with respect to the interaction between rainfall and potential subgrade swell. First, the harmful characteristics of soil swell manifest themselves only when the moisture content of the subgrade experiences a significant change. This change may or may not occur, depending on the overall conditions. For example, the Houston area has swelling subgrades, but, since the moisture content is always high, the swell/shrink cycles are considerably less harmful to the pavements than those experienced in areas where the moisture content changes are greater. Second, the available CRCP data indicate the existence of construction and design practices tend to compensate for these soil expansion effects. For example, the best subbases are always found in test sections located in areas subject to high potential subgrade swell and high rainfall. On the other hand, some influence of the interaction between subgrade swell and amount of rainfall on the distress manifestations was detected in Ref 6, and it may have been caused by foundation support conditions.

TABLE 2.3. CLASSIFICATION OF K-VALUES FROM LAYERED THEORY INTO SWELL*RAINFALL GROUPS

Actual Group	H2	H3	H4	L1	L2	L3	L4	Other	Totals
H2	36.0	32.0	0.0	0.0	8.0	4.0	4.0	16.0	100
H3	20.0	0.0	8.0	0.0	0.0	32.0	0.0	40.0	100
H4	0.0	0.0	57.1	0.0	0.0	14.3	10.7	17.9	100
L1	0.0	2.7	0.0	75.7	0.0	13.5	0.0	8.1	100
L2	11.5	3.9	3.9	0.0	30.8	30.8	0.0	19.2	100
L3	4.6	11.3	0.0	4.6	4.6	59.1	0.0	16.0	100
L4	11.1	0.0	22.2	0.0	0.0	0.0	33.3	33.3	100
% of Total	10.3	7.7	10.8	15.5	6.2	26.8	3.6	19.1	100

TABLE 2.4. CLASSIFICATION OF K-VALUES FROM PLATE THEORY INTO SWELL*RAINFALL GROUPS

Actual Group	H2	H3	H4	L1	L2	L3	L4	Other	Totals
H2	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	100
H3	0.0	25.	6.3	0.0	6.2	31.3	0.0	31.2	100
H4	0.0	0.0	85.7	0.0	0.0	0.0	10.7	3.6	100
L1	0.0	2.9	0.0	82.4	2.9	5.9	0.0	5.9	100
L2	0.0	6.7	0.0	0.0	13.3	40.0	0.0	40.0	100
L3	0.0	17.9	7.1	0.0	3.6	35.7	0.0	35.7	100
L4	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	100
% of Total	0.0	8.7	26.0	22.0	3.9	18.1	2.4	18.9	100

CHAPTER 3. CRITICAL REVIEW OF THE INPUTS FOR THE CRCP MODELS

INTRODUCTION

As discussed in Chapter 1, the performance of CRC pavements is a function of the following variables:

$Z_i - Z_c$ = initial – current distress index,
D = slab thickness,
E_c = elastic modulus of the PC concrete,
k = modulus of reaction on top of the subbase,
J = load transfer coefficient,
C_d = drainage coefficient,
W_{eq} = cumulative equivalent single axle loads applications, and
f_f = flexural strength of PC concrete.

Although indication about some of the input data sources has been given in previous chapters, a critical review of the available inputs for the model seems imperative, not only because the accuracy of any model depends on that of its inputs but also because a number of different sources of data had to be used to develop the necessary input.

DISTRESS INDEX (Z)

Theoretically, the implementation of a pavement management system (PMS) requires a decision criteria index (DI) and a rehabilitation criteria index (RCI), such that the time for overlay is reached when inequality (Eq 3.1) holds:

$$DI > RCI \tag{3.1}$$

where

DI = decision criteria index, and
RCI = rehabilitation criteria index.

Ideally, these indices should comprise pavement condition, safety, and economic factors; however, the state-of-the-art is still limited to the implications of pavement distress in the DI. As for the RCI, it usually consists of a threshold value for DI. AASHTO's Present Serviceability Index (PSI) is an example of a decision criteria index, and its level of acceptability corresponds to the rehabilitation criteria index. PSI depends primarily on roughness, and, since it is not a good indicator of Texas CRCP condition, development of a more representative parameter was imperative.

The index Z developed in Ref 6 consists of an improvement upon a distress index developed earlier (Ref 13). It was derived from a discriminant function (Eq 3.2), calibrated with data from the available condition survey data base, with the objective of separating the pavements into two distinct groups: overlaid and non-overlaid:

$$Z_i = C + \sum_{i=1}^{m} a_i z_{ij} \tag{3.2}$$

where

i = 1, n;
j = 1, m;
Z_i = discriminant score of the i^{th} datum;
a_i = coefficient;
z_{ij} = standardized value for the i^{th} discriminant variable (distress measure) used in the analysis;
n = number of data; and
m = number of discriminant variables.

After some algebraic transformations, which were made primarily to obtain an index that decreases as the pavement condition deteriorates, the final distress index was defined by (Ref 6)

$$Z = 1.0 - 0.0071 * MPO - 0.3978 * SPO - 0.4165 * PAT \tag{3.3}$$

where

Z = distress index,
MPO = ln (number of minor punch-outs per mile + 1),
SPO = ln (number of severe punch-outs per mile + 1),
PAT = ln (number of patches per mile + 1), and
$\ln$ = natural logarithm.

In Eq 3.3, the smaller the value of Z, the better the pavement. Z = 0 defines the boundary between the two groups, assuming that there is equal probability of misclassifying non-overlaid pavement and overlaid pavements. Another assumption used in the statistical method employed in Ref 6 is the normality of the distributions of the discriminant variable in both groups. A normally distributed variable varies from minus infinity to plus infinity, and this cannot be valid for pavements. Since, at early ages, the pavement is at its best condition, and it deteriorates with time and traffic, the distribution of any variable capturing pavement condition must have a high boundary—the value corresponding to the best condition. Therefore, the results of the discriminant analysis performed in Ref 6 are affected by the consequences of the failure in the assumption of the normality of the underlying distributions.

Another important limitation is inherent in the data from which the distress index (Z) was obtained. In the calibration process of the discriminant function (Eq 3.2), the existent 12-year condition survey data base was used. The data collection procedures used in those periodic surveys were adjusted according to previous experiences, practical constraints, and particular research needs of the time each survey was undertaken (Refs 6 and 10). The changes in data collection from survey year to survey year were handled by a data reduction process through which a uniform and consistent series of distress data values was extracted from the data base. This resulted in the availability of data only on the number of punch-outs and patches. Consequently, the Z-score classifies as equally deteriorated a pavement section having, for example, one very large patch and another having one very small patch. For the same reason, in consecutive surveys, the Z-score sometimes indicates false improvement of the pavement condition, especially for situations such as a CRCP section that had several neighboring punchouts covered by a single patch. Despite the practical importance of this limitation, its removal is almost impossible, because it would require the calibration of a new discriminant function with a more appropriate long-term condition survey data base, which is infeasible to obtain, not only due to time and cost constraints but, especially, because the test sections are continuously being overlaid. The course of action taken to partially avoid effects of this limitation in the model is discussed in Chapter 4.

LOAD TRANSFER COEFFICIENT (J)

The load transfer coefficient is a dimensionless number that attempts to define the amount of load that can be transferred from one side of a pavement discontinuity to another. Conceptually, it can be viewed as a correction factor for the pavement stress, as shown in Spangler's equation

$$\sigma_s = \frac{J.P}{D^2}\left(1 - \frac{\sqrt{2a^2}}{L}\right) \tag{3.4}$$

where

σ_s = stress in the PC concrete,
J = load transfer coefficient,
P = wheel load,
D = slab thickness,
a = radius of contact area, and
L = radius of relative stiffness.

During the development of the AASHO Road Test equation, the J-value in Eq 3.4 was set to 3.2, to represent the corner load condition, which was the critical case for the pavement sections in the AASHO Road Test. Later, during the development of the AASHTO Guide (Refs 1 and 2), the conditions at the AASHO Road Test were extended to other conditions (Ref 2). Mechanistic solutions for the stress in the PC concrete slab were obtained by a discrete element program that calculates stresses and strains in an orthotropic slab resting on a spring foundation (Ref 15). The basic assumption made was that equal ratios of strength/stress would give equal performance (Ref 2), i.e.,

$$\sigma_s / \sigma_m = C1 \tag{3.5}$$

where

σ_m = stress from mechanistic analysis,
σ_s = stress from Spangler's equation, and
C1 = constant.

Using a prime for conditions other than those at the AASHO Road Test and letting C2 be the value obtained by inserting in Eq 3.4 all variables, except J, left as unknown, it follows that

$$\sigma_s' = J\,C2 \tag{3.6}$$

Substituting stresses for a condition other than AASHO Road Test in Eq 3.5 and setting σ_s in Eq 3.5 equal to σ_s' in Eq 3.6, J becomes

$$J = (\sigma_s\, \sigma_m') / (C2\, \sigma_m) \tag{3.7}$$

where all variables are as stated previously.

Table 3.1 depicts the load transfer coefficients obtained with this methodology (Ref 2), and Table 3.2 depicts the design ranges recommended by the AASHTO Guide (Ref 1).

TABLE 3.1. DERIVED LOAD TRANSFER COEFFICIENTS FOR CRCP (REF 2)

Conditions		J-Values for Shoulder Type	
Thickness (in.)	k (pci)	Flexible	Tied Rigid
7	100	2.9	2.5
10	100	3.0	2.6
13	100	3.1	2.6
7	600	2.6	2.3
10	600	2.8	2.4
13	600	2.9	2.5

Currently, there is no standard procedure to obtain the J-factor of an existing pavement. Reference 1 suggests that each Agency develop its own factors. This recommendation is especially important for this study, because its primary purpose is to obtain a CRCP model capable of reflecting local conditions.

TABLE 3.2. RECOMMENDED LOAD TRANSFER COEFFICIENTS (REF 2)

	Flexible Shoulder		Tied Rigid Shoulder	
Pavement Type	LTD	No LTD	LTD	No LTD
Plain or Jointed	3.2	3.8 to 4.4	2.5 to 3.1	3.6 to 4.2
Continuously Reinforced	2.9 to 3.2	–	2.3 to 2.9	–

Direct use of the approach described above for determining J-factors requires that stress measurements be available. A usual surrogate variable for distress is deflection, which was collected for the CRCP test sections during the diagnostic survey. Reference 45 developed a procedure for estimating J-factors, based on the principles described above, and using deflection data to develop a relationship between the field conditions and the theoretical conditions. Sensitivity of load transfer conditions to FWD deflections is supported by experimental evidence (Ref 28). The following assumptions underlie the procedure developed in Ref 45:

(1) Westergaard's equations (Ref 48) are valid models for pavement response due to different load positions in the panel,

(2) interior load represents 100 percent load transfer conditions,

(3) effects of the temperature gradient in the edge deflections are negligible, and

(4) differences between measured edge deflections and those calculated with Westergaard's equations are due to the effect of load transfer in the field and are in the same ratio as the deflection ratio, as shown in

$$\sigma' = \frac{\sigma_c\, d'}{d_c} \qquad (3.8)$$

where

σ = stress,
d = deflection,
subscript c = calculated by mechanistic equations, and
prime = field conditions.

Ratios of calculated to measured edge deflections were used to modify the load and obtain the stress at the edge. Next, results from loop 1 of the AASHO Road Test were used in conjunction with Spangler's equation to obtain the J-factors. The stress for loop 1 of the AASHO Road Test is (Ref 45)

$$\sigma_{loop1} = \frac{0.32\,P}{\sqrt[3]{D^4}} \qquad (3.9)$$

where

P = load and
D = slab thickness.

The model used for obtaining the load transfer coefficients (Ref 45) is

$$J = 3.2\,\frac{\sigma'\,\sigma_{sp\text{-}loop1}}{\sigma_{loop1}\,\sigma'_{sp}} \qquad (3.10)$$

where

σ' = field stress,
$\sigma_{sp\text{-}loop1}$ = stress computed with Spangler's equation for loop 1 conditions,
σ_{loop1} = loop 1 stress, and
σ'_{sp} = stress computed with Spangler's equation for field conditions.

The J-values obtained with this approach are summarized in Fig 3.1. Assumption 3 is not supported by experimental results (Ref 28), but it had to be made at this stage of the study due to the lack of a procedure for correcting deflection measurements to a common temperature. However, an ongoing effort in this project is attempting to develop such a procedure. Should this effort be successful, it is suggested that the model

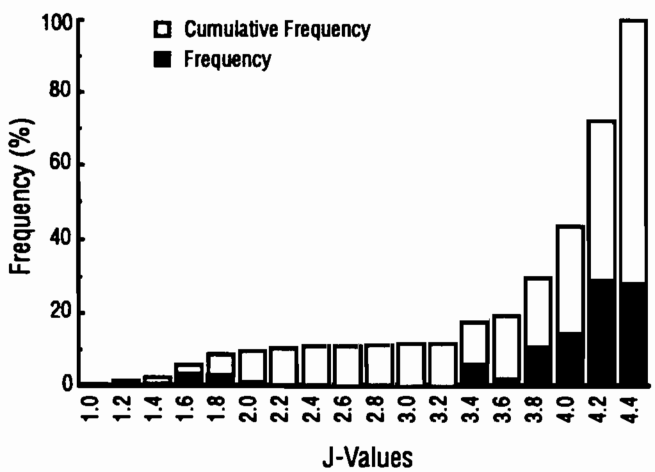


Fig 3.1. Load transfer coefficients for CRCP test sections.

discussed in the next chapter be recalibrated with these new J-values in a later stage.

DRAINAGE COEFFICIENT

The drainage coefficient (C_d) was introduced in the new AASHTO Guide (Refs 1 and 2) to account for the positive impact of good drainage on pavement behavior. The effects of water in the rigid pavement layers include (Ref 2):

(1) ***PC Concrete Slab.*** Although moisture has some effect on the strength and elastic modulus, the main effect of added moisture is caused by stresses induced by restrained warping and curling due to moisture differentials along the slab.

(2) ***Granular Subbases.*** Reductions in modulus values of more than 50 percent, due to added moisture, have been reported in the literature.

(3) ***Treated Subbases.*** For asphalt treated layers, modulus reductions of up to 30 percent can be expected; for cement or lime treated, reduction is slight and some erosion can be expected.

(4) ***Roadbed Soil.*** Permeable soils are not expected to be subject to considerable modulus reductions, while those with low permeability can experience reductions of up to 50 percent.

The AASHTO model for designing rigid pavements has the following format (Ref 1):

$$\log(W_{eq}) = Z\alpha\, S_O + 7.35 \log(D+1) - 0.06 + \frac{\log\left[\dfrac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \dfrac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$(4.22 - 0.32\, p_t) \log\left[\frac{f_f\, C_d\, [D^{0.75} - 1.132]}{215.63\, J\left[D^{0.75} - \dfrac{18.42}{\left[\dfrac{E_c}{k}\right]^{0.25}}\right]}\right] \qquad (3.11)$$

where

W_{eq} = cumulative equivalent 18-kip single axle load applications,

Z_a = standard normal deviate, for a probability a,

S_0 = combined standard error of the traffic prediction and performance predictions,

ΔPSI = initial - final PSI,

p_t = terminal PSI,

D = slab thickness (inches),

f_f = flexural strength of PC concrete (psi),

J = load transfer coefficient,

C_d = drainage coefficient,

E_c = modulus of elasticity of PC concrete (psi),

k = modulus of subgrade reaction (pci), and

logs = base 10.

It can be seen that C_d was introduced in the numerator of the portion of the performance equation that considers the slab strength and support conditions. The coefficients show that C_d was given the same relative importance as the flexural strength of the PC concrete and the load transfer coefficient.

The approach used in Ref 2 to obtain the values of C_d was to back-calculate them from Eq 3.11, for a range of different conditions that had a significant impact on thickness. The reliability term was not used in the calculations. At this stage of research in CRCP performance, this approach requires a considerable number of assumptions to be made, because a reliable performance prediction model is needed to estimate the drainage coefficients, but the latter are needed to calibrate a performance prediction model. In an effort to overcome this problem, (Ref 36) developed a regression equation that estimates C_d as a function of rainfall and subbase type. The model is

$$C_d = 2.171 - 0.0149 * RAIN + SBT \qquad (3.12)$$

where

C_d = drainage coefficient;

RAIN = average annual rainfall, in inches;

SBT = –0.3649 (for asphalt treated subbases);
–0.2784 (for cement-treated subbases); and
–0.4641 (for crushed stone subbases).

Perhaps the most important simplifying assumptions that had to be made to arrive at these coefficients relate to the PSI-values and to the J-values used in Eq 3.11. Since PSI-values were not available, a linear correlation was assumed between PSI and Z-score. As discussed before, PSI is not a reliable indicator of Texas CRC pavement performance, whereas Z seems to be. Thus, any correlations between Z and PSI are in disagreement with all the previous findings concerning performance of the test sections used in the calculations. Load transfer coefficients (J) were assumed equal to 3.2. This does not seem a realistic assumption and, given the relative impact of J in Eq 3.11, it may have caused important errors in C_d estimates.

An alternative procedure for estimating the drainage coefficients based on the recommended values in Ref 1 can theoretically be used. Those values are a function of percent of time the pavement is expected to experience saturation and of the quality of the drainage, which is a function of the porosity and the permeability of the

TABLE 3.3. RECOMMENDED DRAINAGE COEFFICIENTS (REF 1)

Quality of Drainage	Percent of Time the Pavement Structure Approaches Saturation			
	< 1	1 to 5	5 to 25	> 25
Excellent	1.25 to 1.20	1.20 to 1.15	1.15 to 1.10	1.10
Good	1.20 to 1.15	1.15 to 1.10	1.10 to 1.00	1.00
Fair	1.15 to 1.10	1.10 to 1.00	1.00 to 0.90	0.90
Poor	1.10 to 1.00	1.00 to 0.90	0.90 to 0.80	0.80
Very Poor	1.00 to 0.90	0.90 to 0.80	0.80 to 0.70	0.70

TABLE 3.4. QUALITY OF DRAINAGE (REF 2)

Quality of Drainage	Time Period for Water Removal	
	Calculated	Recommended
Excellent	2 to 4 hours	2 hours
Good	0.5 to 6 days	1 days
Fair	3 to 6 days	7 days
Poor	18 to 36 days	30 days
Very poor	> 30 days	Does not drain

TABLE 3.5. TIME TO DRAIN BASE LAYER TO 50 PERCENT SATURATION (REF 2)

Permeability, ft/day	Porosity	Slope	H = 1		H = 2	
			L = 12	L = 24	L = 12	L = 24
0.1	0.015	0.01	10	36	6	20
		0.02	9	29	5	18
1.0	0.027	0.01	2	6	5	18
		0.02	2	5	1	3
10.0	0.048	0.01	0.3	1	0.2	0.6
		0.02	0.3	1	0.2	0.6
100.0	0.08	0.01	0.05	0.2	0.03	0.1
		0.02	0.05	0.2	0.03	0.1

H = Thickness of drainage layer, ft
L = Length of drainage path, ft

layers. Tables 3.3, 3.4, and 3.5 present those recommendations. Permeability values need to be determined in the laboratory, but ranges of values for several types of materials can be found in the literature (Refs 12, 17, 26, 38, and 29). Porosity can be calculated as a function of field dry density (or dry unit weight) and true density (or true unit weight):

$$n = e/(1+e) \tag{3.13}$$

and

$$\gamma_d = \gamma_a/(1+e) \tag{3.14}$$

where

n = porosity,
e = void ratio,
γ_d = dry unit weight or density (gravitational acceleration cancels out), and
γ_a = true unit weight or density (gravitational acceleration cancels out).

True densities range from 2.6 kg/dm³ (0.094 pci) to 2.8 kg/dm³ (0.101 pci) for most soils (Ref 38). Since the percents of stabilizing materials eventually used are very low, it is reasonable to use an average value of 2.7 kg/ dm^3 (0.097 pci) for the true density, or 26.48 KN/m^3 unit weight for the standard gravitational field. Field dry densities at the end of construction should ideally be obtained from construction records. If these are not available, some rationale is needed to obtain estimates comparable on a common basis, because field densities are unlikely to remain unchanged with time.

If some procedure for estimating the percent of time the pavement section is subject to saturation could be developed, C_d values could be estimated from the ranges in Table 3.3.

The procedure outlined above does not eliminate the problem of using C_d values from the AASHTO performance prediction model to calibrate another model, because of the origin of Table 3.3. However, the AASHTO Guide (Ref 2) does not make use of specific pavement sections, whereas Shyam (Ref 36) arrived at C_d values using data from, and making several assumptions about, specific CRCP sections that will also be used to calibrate the model. On the other hand, the procedure suggested here either relies on the undesirable practice of estimating data from typical ranges found in literature, or requires further data collection and laboratory tests, which do not seem feasible in a network level basis and would not provide accurate estimates of field density. This problem seems worthy of further investigation, especially if the contribution of available C_d values to the model turns out to be non-significant.

EQUIVALENT SINGLE AXLE LOADS (ESAL)

Simultaneously with the 1987 condition survey described in Ref 6, a major effort was being made by Project 1169 to obtain traffic data from the SDHPT records (Ref 37). Because of the particular needs of Project 1169, traffic data was obtained only for overlaid sections. Facilities, such as counting and weigh stations, are not assigned by agencies to suit specific the needs of a research project. For example, there are no weigh stations in urban areas. Consequently, the best that can be done is to develop some rationale for assigning available traffic data to the experimental sections. The procedure applied to assign ESAL data to the test sections in the CRCP data base is

(1) directly assign ESAL data from Project 1169 sections (Ref 37) and
(2) estimate ESAL data based on data from Project 1169 sections.

Since Project 1169 test sections (Ref 37) were selected from the data base being used by this study, step 1 is relatively straightforward, and it is the most accurate. To every matching test section, ESAL data were assigned either from the closest Project 1169 section, or from the average of all Project 1169 sections within the pertinent section length used in this project. Both procedures yielded very similar results, because few study lengths in this project have more than one Project 1169 section, and, for those that have, the differences between available traffic data within the same study length were negligible. This similarity may be due to the fact that, since the study lengths range from 1 or 2 miles to no longer than 15 or 16, the presence of an exit or a junction between two Project 1169 sections in the same study length is not very frequent. Ref 37 describes in detail the procedure for obtaining average daily traffic (ADT), percent trucks, percent tandem axles, and ESAL for Project 1169 test sections. The following sources, all available at the State Department of Highways and Public Transportation (SDHPT), for manual retrieval, were used in Ref 37 to obtain traffic data and its location in the road:

(1) ADT maps,
(2) road inventory file, and
(3) traffic logs for each district.

An important practical consideration of this procedure is that it is not cost effective, because it requires a considerable amount of work time from someone qualified to make subjective decisions. This is true especially for the ESAL. Data on ADT, percent trucks, average ten heaviest wheel loads, and percent tandem axles are considerably less time consuming to obtain from SDHPT. This fact motivated an attempt to use data from Ref 37 to model ESAL as a function of the above mentioned data. Dossey and Weissmann (Ref 9) developed several models of this kind that gave a remarkably good fit. However, since equivalent single axle loads depend not only on traffic but also on pavement load carrying capacity (Refs 1, 14, 44, and 49), findings in Ref 9 were used as a starting point to arrive at the following model:

$$\ln(\mathrm{ESAL2}) = 0.047\ \mathrm{PTRUCK} + 0.9389 \ln(\mathrm{ADT}) + 0.236\ \mathrm{D} + 0.0018\ \mathrm{ATHWL} \quad (3.15)$$

where

ESAL2 = two-direction equivalent single wheel load;
PTRUCK = percent trucks in ADT;
ADT = average daily traffic
ATHWL = average ten heaviest wheel loads; and
D = slab thickness, inches.

The R2 of this model is 99.99 percent, the smallest significant level of the regression parameters is 0.0001, and the significance level of the overall model is 0.0001. Figure 3.2(a) is a plot of the predicted versus actual values, and Fig 3.2(b) is the residual plot of the model shown in Eq 3.15, which was used to obtain equivalent single axle loads (ESAL) for the remaining test sections for every survey year.

In summary, there are two types of ESAL data in the CRCP data base:

(1) that obtained by Shyam (Ref 37) directly from SDHPT records and
(2) that obtained through Eq 3.15, whose inputs also come from SDHPT records.

Apparently, the most important drawback of both procedures is the non-correspondence of test sections with traffic counters and weigh stations. The SDHPT sections are considerably longer than the test sections under study; they may well encompass junctions, exits, and other facilities that certainly interfere with traffic. Those limitations are even more accentuated for truck data, due to the small number of weigh stations statewide. Currently, procedure 1 is the best that can be done to assign ESAL data for any pavement section in Texas.

PAVEMENT THICKNESS

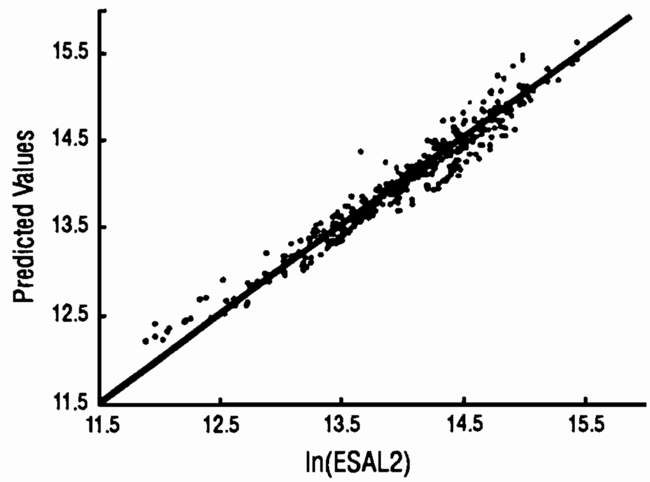
Slab thickness is available in the data base, and it was obtained from SDHPT records. Since it is evident that any considerations of the variability of thickness are irrelevant as compared to the already discussed sources of errors in the other variables, no further discussion seems applicable here.

SUMMARY

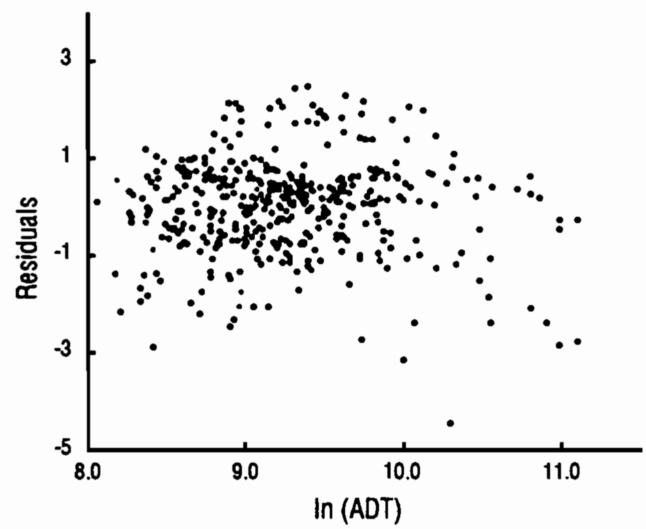
This chapter discussed the limitations of the candidate explanatory variables for the performance prediction model. These limitations can be summarized as follows.

(1) The distress index considers only the number of punchouts, the number of patches, and two categories of punchout severity. Relative sizes of patches are not captured by the distress index equation, and false improvements in pavement condition can be inferred from the analysis of condition survey data using the distress index.
(2) The elastic modulus of the PC concrete, the modulus of reaction on top of the subbase, and the load transfer coefficient were obtained through back-calculation from deflection data, and thus they have all the limitations and drawbacks inherent in this practice. Lack of accurate pavement temperature data introduced additional errors in the estimated load transfer coefficients, because the coefficients are back-calculated using edge deflections, which are affected by pavement temperature differentials.
(3) The flexural strength of the PC concrete has to be estimated using an empirical relationship with the elastic modulus. The validity of the use of this type of estimate as an explanatory variable in a model can be dubious, because the errors of the back-calculated elastic modulus will superimpose on the errors inherent in empirical relationships.
(4) The drainage coefficients are very difficult to estimate at this stage of the research study in CRCP performance, because a performance model is needed to reliably estimate these parameters from field data. Another option is to collect several types of data relating to the drainage conditions of the pavement, but the practical disadvantages of this approach are very likely to prevent its practical application.
(5) The traffic data is perhaps the most inaccurate of the model variables, and elimination of these inaccuracies are very difficult, if not impossible, to attain in practice. The ideal situation is to count traffic and to weigh vehicles in each test section, while in real life these data have to be extrapolated from a few counting and weighing stations scattered over the state.

In the calibration process, it is possible that some variables may not significantly help explain the variations in the dependent variable. On the other hand, other variables may be added to the model if it appears that their addition will be reasonable from a technical point of view. Some examples of the variables available in the data base that might be good candidates for inclusion in the model are presence or absence of swelling soil and average annual rainfall. Sources of data for these variables are described in Refs 6 and 10.



(a) Predicted versus observed values.



(b) Residuals.

Fig 3.2. Results of the calibration of the model for predicting ESAL.

CHAPTER 4. A SURVIVAL MANUAL FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

PROBLEM DEFINITION

FAILURE AND ACCEPTABILITY OF A CRC PAVEMENT

The typical design problem for civil engineering structures is usually attached either by an approach that seeks to prevent sudden failure of the structure due to static overload or by an approach that takes into account the fatigue life of the materials, i.e., by designing the structure for a number of stress cycles. Perhaps because pavement failures are not spectacular and cause deaths only indirectly, the concept of failure is much more vague for pavements than it is for other structures. For example, a given number of stress cycles can be defined as the fatigue life of a beam, because the beam may crack and fall apart if loaded after having supported that number of stress cycles. In the case of a pavement, a given number of stress cycles may cause cracks, but pavements can carry traffic despite cracks. What is life-length, or fatigue life, in this case? According to Ulidtz (Ref 44), a proper definition of life-length is

> the number of wheel passages of a specified type that the pavement can support before it reaches an unacceptable level of functional or structural distress.

In the definition above, the vagueness is only transferred from the concept of failure to the concepts of acceptability of functional and structural distress. A widely used textbook on pavement design (Ref 49) defines structural and functional failures as follows:

(1) Structural failure is a collapse of the pavement structure, or a breakdown of one or more of the pavement components, of such magnitude to make the pavement incapable of sustaining the loads imposed upon its surface.
(2) Functional failure, which may or may not be accompanied by structural failure, is such that the pavement will not carry out its intended function without causing discomfort to users or causing high stresses in the vehicles passing over it.

The definitions above also depend on the definition of the acceptability of a pavement, a very important subjective concept. The AASHO Road Test represents one of the most serious efforts to eliminate the vagueness of this definition. Its performance models and its failure criterion are based on the present serviceability index (PSI), which is basically a function of roughness. Since the PSI is not a good indicator of the CRCP distress condition (see Figs 1.2 and 1.3), the AASHTO concepts of pavement failure and acceptability do not give good results when applied to CRC pavements.

The CRCP test sections used in this study are subject to an excellent routine maintenance that keeps the riding quality as good as possible. These pavements are usually overlaid when the cost of the routine maintenance becomes as high as the overlay cost. The failure, or acceptability criterion, embedded in the CRCP distress index, is the cost of routine maintenance. The underlying definition of failure used in this study can be stated as follows:

> A CRC pavement fails when the costs of the routine maintenance required to keep its riding quality good are equal to the costs of an overlay.

Since the overlay date is assumed to be the failure date, another underlying assumption embedded in the development of the distress index Z is that the decision to overlay is made only on the basis of maintenance costs and no other factors ever affect this decision. This assumption also affects the models developed in this study.

The calibration of a performance model also requires that the variable to measure life-length be determined. In most structural engineering problems, the life-length of a structure is measured in number of stress cycles. For pavements, the stress cycles are measured in terms of number of passages of a standardized single axle, and this is perhaps the only aspect of pavement design that is not controversial. Several criteria have been developed to convert the actual vehicles into the standard vehicle for design purposes (see, for example, Refs 44 and 49). In this study, the AASHTO equivalency (Refs 1 and 2) is used, in order to be consistent with current design practices.

IDEAL FEATURES OF A PERFORMANCE PREDICTION MODEL

The problems of designing and structurally evaluating a pavement are generally tackled using one of the following basic approaches:

(1) Mechanistic, which solves a simplified physical model of the structure, either through closed form solutions of theoretical governing equations (e.g., Ref 48) or by numerical approximations (e.g., Ref 15).
(2) Empirical, in which a set of experimental data is used to calibrate an equation relating the variables. The CBR method (Ref 49) and the AASHTO equation (Ref 1) are classical examples of this approach.

In both approaches, a specific estimate of the variable of interest is sought, and the uncertainty is usually dealt with by means of empirical safety factors. Recently, however, the attention of engineers and practicers has broadened to consider the uncertainty inherent in the process in probabilistic terms. The efforts of the AASHTO Task Force to determine the reliability of a pavement design illustrate this new approach (Refs 1 and 2). Within this new framework, a performance model should actually seek the probability that a pavement section will survive a given limit. A more precise way of posing this fundamental question is the following:

> What is the probability that a pavement section will not fail at a given number of equivalent single axle load applications, conditioned to a specific set of values for the concomitant variables that characterize the section?

The problem is thus to determine either

$[x_1, x_2, x_3 ..., x_n]$ such that

$$P[(W_{eq} \geq W_{eq0}) / x_1, x_2, x_3 ..., x_n] = Q \tag{4.1}$$

or

W_{eq} such that

$$P[(W_{eq} \geq W_{eq0}) / x_1, x_2, x_3 ..., x_n] = Q \tag{4.2}$$

where

W_{eq} = cumulative number of equivalent single axle load applications,

W_{eq0} = distribution quantile of W_{eq},

P(A/B) = probability of event A conditioned on B,

Q = probability associated with quantile W_{eq0}, and

$[x_1, ..., x_n]$ = vector of concomitant variables.

The determination of the concomitant variables $[x_1, ..., x_n]$ that satisfy Eq 4.1 can be viewed as the design problem, whereas the determination of the equivalent single axle load applications (W_{eq}) that a given pavement can carry is the evaluation problem.

APPROACH FOR ANALYZING THE DATA

INTRODUCTION

The approach chosen for calibrating a model of the general format described in Eqs 4.1 and 4.2 is to assess the relationship between failure time and concomitant variables using Survival Analysis, a collection of statistical techniques for analyzing life-length data (Refs 5, 11, 16). This method was adopted after careful literature review; it is the most appropriate method for arriving at the desired results, without major failures in the underlying assumptions of the selected statistical methods.

BASIC CONCEPTS AND TERMINOLOGY OF SURVIVAL ANALYSIS

The calibration of empirical pavement models has always been done using least squares regression techniques, and applications of Survival Analysis in pavement engineering are still unexplored. Therefore, it is convenient to summarize the basic concepts and terminology used in Survival Analysis before discussing its application to this research study. The summary below is based on Refs 5, 11, and 16.

(1) ***Survivor Function.*** A common concept of statistics is the cumulative distribution function of a random variable T, usually represented by F(T). It gives the probability that the random variable has a value that is less than a given t. In applications of Survival Analysis, however, the random variable usually represents the failure time of an individual from a population, and the main interest is to determine the probability that the random variable has a value that is greater than some limit t. This function is termed either survivor function (Refs 11 and 16) or reliability function (Ref 5) and is represented by S(T). Its relationship to the cumulative distribution function is shown in Fig 4.1 and in Eq 4.3. In order to avoid confusion with the reliability of the AASHTO model (Refs 1 and 2), which is not based in the calibration of a survivor (or reliability) function, the term survivor function is used in this chapter:

$$S(T) = 1-F(T) = P(T \geq t) \tag{4.3}$$

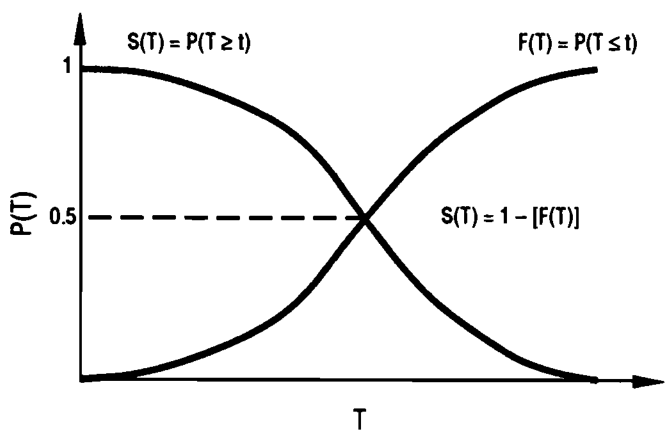


Fig 4.1. Concept of survivor function.

(2) ***Hazard Function***. In many applications of Survival Analysis, some information is available on how the failure rate changes with the failure time. This information is captured by the hazard function, defined by

$$H(T) = \frac{1}{1 - F(T)} \frac{d\,F(T)}{d\,T} = \frac{d\,[\ln S(T)]}{dT} \qquad (4.4)$$

where ln represents the natural logarithm and all other terms are as defined previously.

(3) ***The Weibull Distribution***. Among the theoretical distributions for modeling failure time, the Weibull distribution seems the most appropriate for CRC pavements, because its hazard function captures a monotonic dependence of the rate of failure on the time variable. In other words, it can represent the anticipated fact that, the older the pavement, the less likely it is to survive. The Weibull distribution is

$$f(T) = \lambda\, p\, (\lambda T)^{p-1} \exp\,[-(\lambda T)^p] \qquad (4.5)$$

where

$f(T)$ = probability density function of T,
λ = scale parameter,
p = shape parameter, and
$\exp(X) = 2.7182818^X$.

The hazard function of a Weibull distribution is

$$H(T) = \lambda\, p\, (\lambda T)^{p-1} \qquad (4.6)$$

and the survivor function is

$$S(T) = \exp\,[-(\lambda T)^p] \qquad (4.7)$$

where all parameters are as in Eq 4.5.

Figure 4.2 shows the possible shapes of hazard function of a Weibull distribution (Eq 4.6). Depending on the value of the shape parameter, the failure rate can either increase or decrease with the failure time variable. For $p = 1$, the rate of failure does not change with time, and the corresponding distribution function is the exponential, which does not feature a shape parameter, and is a special case of the Weibull distribution.

It is clear from Eq 4.7 that a goodness-of-fit test for a Weibull distribution is provided by the plot of the logarithm of the negative of the logarithm of the survivor estimates versus the logarithm of the survival time variable, because

$$\ln[-\ln \hat{S}(T)] = p\,(\ln T + \ln \lambda) \qquad (4.8)$$

where S(T) is the sample estimate of the survivor function and the other parameters are as defined in Eqs 4.6 and 4.7.

REGRESSION MODEL FOR SURVIVAL TIME DATA

Consider a failure time variable T and a vector of covariables $\mathbf{x}=[x_1, x_2, x_3, ..., x_n]$, which may include categorical variables. The problem is to determine the relationship between T and **x**, given a baseline distribution for T. Where T follows a Weibull distribution, the model is (Ref 16)

$$(\ln T)_q = A + \mathbf{x}\,\mathbf{B} + \sigma\,\omega_q \qquad (4.9)$$

where

ln = natural logarithm,
T = failure time variable,
$(\ln T)_q$ = q^{th} quantile of the survivor function of ln(T),
A = intercept,
x = vector of covariables,
B = vector of regression coefficients,
σ = scale parameter, and
ω_q = q^{th} quantile of the Type 1 asymptote of minima, tabulated in Appendix D.

The model depicted in Eq 4.9 estimates the survivor distribution conditioned on a given set of covariable values, represented by **x**. In other words, it shifts and rescales the baseline distribution, according to the values assumed by the concomitant variables. Figure 4.3 shows this concept, which corresponds to the desired model format depicted in Eqs 4.1 and 4.2.

COMPARISON BETWEEN SURVIVAL MODEL AND LEAST SQUARES MODEL

The reasons for the choice of a the survival model of the type depicted in Eq 4.9 can be better explained through a comparison with ordinary least squares regression models.

The survival models assume that the dependent variable follows a previously fitted distribution, that is shifted and re-scaled, depending on the values of the concomitant variables. In other words, for each value of the vector of concomitant variables, a survival model provides an estimate of the survivor function of the dependent variable. Figure 4.3 shows a conceptual form of a survival model.

The least squares method assumes that, for each value of the vector of explanatory variables, there is an independent normal distribution of the dependent variable whose mean lies on the fitted regression line and whose variance is constant for all independent distributions. For each value of the vector of explanatory variables, a least squares model provides a specific estimate of the dependent variable. Figure 4.4 shows this concept, for a single explanatory variable.

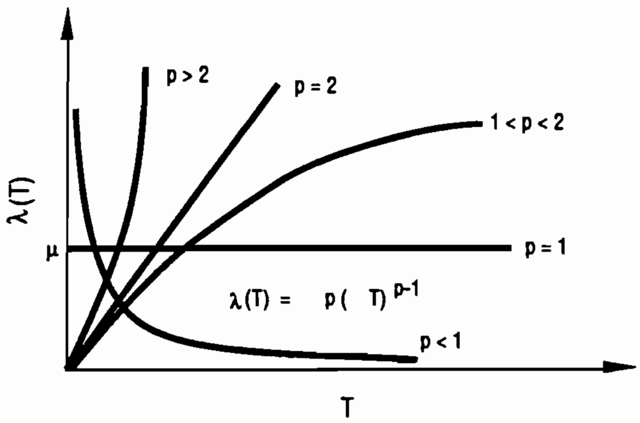


Fig 4.2. Form of the hazard function of a Weibull distribution.

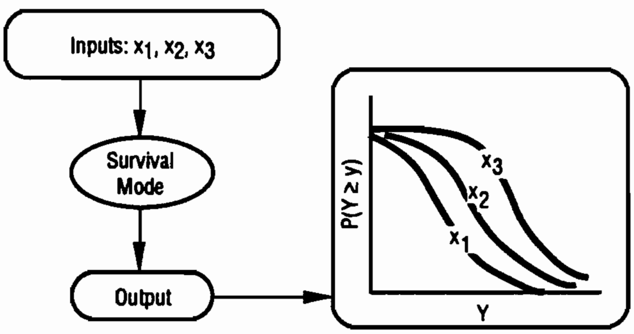


Fig 4.3. Conceptual form of a survival model.

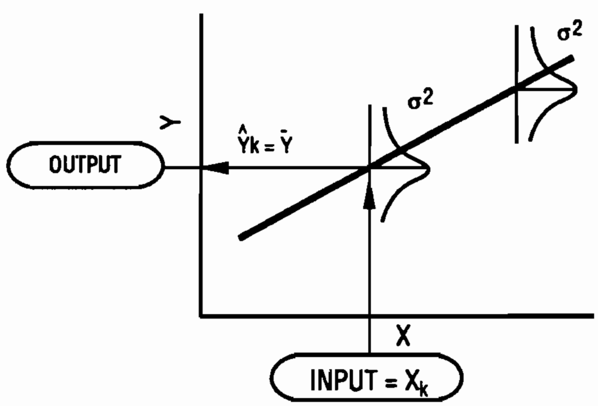


Fig 4.4. Conceptual format of a least squares model.

Figures 4.3 and 4.4 show that the survival model can represent the conceptual model sought in this study (Eqs. 4.1 and 4.2) better than the least squares model. Two other very important advantages of the approach chosen are its better adherence to the underlying assumptions of the statistical method and the capability to consider information provided by censored data.

Although it is assumed that the cumulative number of equivalent single axle load applications (W_{eq}) does not obey the formal mathematical assumptions of the least squares method (for example, W_{eq} values of consecutive years in a test section are not independent), there is a more important assumption that is frequently overlooked: the model format itself.

For example, when a linear model is fitted to predict a random variable Y as a function of X, it is implicitly assumed that the average Y is a linear function of X, at least within the range of values of the available sample.

In this study, the AASHTO model is a possible initial guess for the format of a least squares model of W_{eq} considering the other variables. Since the AASHTO model was calibrated using a wear-out variable (the PSI) and a failure concept that are not adequate for the CRC pavements under study, it seemed too optimistic to hope that this model could be successfully recalibrated for CRCP by substituting PSI for the Z-index. An attempt was made to recalibrate the AASHTO model in this way, but it yielded non-significant results. This fact, together with the other experimental evidence, indicates that the AASHTO model does not explain satisfactorily the deterioration of CRC pavements subject to good maintenance. The only way to arrive at a least squares model would be to make guess after guess of possible model formats. If one format could be found that fits the data, this hypothetical model would still have the following drawbacks:

(1) The model would not provide estimates of the probability of survival, and, more importantly,
(2) The model would be neither theoretically nor statistically sound, first because there would be no guarantee that the fitted format is a satisfactory explanation of the relationship under investigation, and second because most underlying assumptions of the least squares technique would be violated.

Use of survival analysis automatically eliminated the first of the drawbacks. In addition, if a baseline distribution could be satisfactorily fitted to the variable W_{eq}, the second and more important drawback would be drastically decreased.

DATA REDUCTION FOR THE MODEL

GENERAL

As already discussed in previous chapters, the following concomitant variables are primary candidates for inclusion in the model:

$Z_i - Z_c$ = ΔZ = initial – current distress index,
D = slab thickness,
E_c = elasticity modulus of the PC concrete,
K = modulus of reaction on top of the subbase,
J = load transfer coefficient,
C_d = drainage coefficient, and
f_f = flexural strength of the PC concrete.

The following additional concomitant variables are known to have some influence on pavement performance and were also included in the subset of the CRCP data base used for calibrating the model:

RAIN = average annual rainfall,
TEMP = yearly temperature range,
CAT = coarse aggregate type of the PC concrete,
SBT = subbase type, and
SOIL = swelling potential.

A subset of the CRCP data base containing the variables listed above was built for each survey year of each test section, after some transformations and deletions were made to make all the variables meaningful in terms of the desired model.

EQUIVALENT SINGLE AXLE LOAD APPLICATIONS

The variable W_{eq}, the cumulative number of equivalent single axle load applications, is available for each survey year in each test section, regardless of whether or not the section has been overlaid. However, use of this variable to represent failure time in the model implies deletion of some data. The value of W_{eq} corresponding to the first overlay year is the failure time of that particular section. Values of W_{eq} corresponding to non-overlaid sections can be used as censored data, but values of W_{eq} after an overlay has been placed have no meaning in the context of the analysis being undertaken. These data belong to another sample space, that of overlaid pavements, and the analysis of condition survey data in years subsequent to that of the first overlay consists of a study of the overlay performance, not of a study of CRCP performance.

Table 4.1 summarizes the traffic data available to the performance study. The data comprise a total of 747 data points from test sections ranging in thickness from 8 to 13 inches, with minimum W_{eq} of 251,751 and maximum of 133,943,046, at a mean of 10.38 million. The most significant portion of the data lies in the 91 overlaid, or failed, sections, all of which are 8 inches thick. The other 656 test sections are non-overlaid. Even though failure has not yet been reached, the information provided by the 656 non-overlaid sections is useful in Survival Analysis as censored data, as discussed earlier in this chapter.

DISTRESS INDEX

The values of the variable Z, the distress index, had to be made more representative of the condition of the pavement in the modeling framework, where data from the year of first overlay correspond to that of the failure time. In order to make a coherent analysis, the values of Z corresponding to the overlay time must also represent failure. In the CRCP data base, however, the available values of Z for this case usually reflect the results of the survey on the recently overlaid pavement. These were changed to the value that represents the terminal condition, i.e., the value immediately before overlay.

Some time before overlay, it is expected that the pavement will be in a poor condition, which usually means there will be large patches covering up a number of distress manifestations. As discussed in Chapter 3, the Z-score cannot adequately represent this condition, because it is a function of only the number of patches. Consequently, the following corrections were made in the data used for modeling purposes:

(1) The Z-score values corresponding to the year of first overlay were set to the recommended an overlay threshold of zero.
(2) When the Z-score increased in consecutive survey years, it was set to the value of the previous year minus 0.01. The inexplicable improvements in the

TABLE 4.1. SUMMARY OF THE DATA

Type of Test Section	Sample Size	Equivalent Single Axle Loads Mean	Minimum	Maximum
All	747	10,380,000	251,715	133,943,046
Overlaid	91	14,396,608	9,518,166	37,356,723
Non-Overlaid	656	9,823,535	251,715	133,943,046
8-in., Overlaid	91	14,396,608	9,518,166	37,356,723
8-in., Non-Overlaid	609	8,486,658	251,715	41,016,203
9-in., Overlaid	0			
9-in., Non-Overlaid	35	4,750,795	419,624	10,031,432
13-in., Overlaid	0			
13-in., Non-Overlaid	12	92,465,519	36,812,677	133,943,046

Z-score are due to the fact that it captures only the number of distress manifestations, showing false improvement in situations when, for example, several neighboring punchouts are covered by one patch. This deficiency was overcome by transforming all inexplicable improvements into slight worsening.

(3) Whenever the Z-score value in the survey year immediately before the overlay was high, it was set to a value of 0.1, in order to reflect a condition close to the recommended overlay threshold of zero. This correction was made because, immediately before overlay, the section usually presents one or two very large patches, yielding a high Z-score that does not reflect the actual pavement condition.

The corrections explained above help make the performance indicator variable Z-score more sensitive to the actual pavement condition. As new Z-score values are introduced in the data base, similar judgement must be applied to eliminate some of the flaws in the new Z-scores, before using the data in analysis.

LOAD TRANSFER COEFFICIENTS FOR THE OVERLAID SECTIONS

The procedure used to back-calculate load transfer coefficients (J-values) for the CRCP test sections relies on the ratio of deflections near discontinuities to those at the interior of the slab. Therefore, only J-values for non-overlaid sections were available from Ref 45. This required the development of a rationale for assigning J-values to overlaid sections. Since data on slab thickness and modulus of reaction on top of subbase were available, Tables 3.1 and 3.2 in Chapter 3 were used to assign J-values for these test sections. The upper ranges of J-values in Table 3.2 were used because it was assumed that an overlaid section has poor load transfer due to its many discontinuities.

SUMMARY AND ADDITIONAL COMMENTS

The resulting data base subset has 747 observations, which spanned the 14 years of survey data. Of this total, 91 are for overlaid and 656 are for non-overlaid pavements. A sample of the data can be seen in Appendix C, and a summary is in Table 4.1. Lack of variable f_f (flexural strength of the PC concrete) in Appendix C is due to the fact that actual laboratory results are not available for this parameter. The only way to obtain f_f by using an empirical correlation with E_c, shown in Eq 4.10 (Ref 1):

$$f_f = 0.125\left(\frac{E_c}{57{,}000}\right)^2 \tag{4.10}$$

where

f_f = flexural strength, psi; and
E_c = elastic modulus, psi.

It is worth remarking, however, that the meaningfulness of flexural strength values estimated with Eq 4.10, as an explanatory variable in a regression model is questionable, for the following reasons:

(1) Flexural strength estimates from Eq 4.10 are subject to a considerable amount of error. Equation 4.10 is a somewhat rough empirical correlation.

(2) The available E_c values to use in Eq 4.10 were obtained through a procedure also subject to error, thus increasing the amount of error already present in Eq 4.10.

(3) Flexural strength estimates from Eq 4.10 consist of re-scaled values of E_c^2. In other words, fitting a model using E_c and f_f estimates as explanatory variables is mathematically the same as fitting a model in which the contribution of E_c is assumed to be due both to a linear term and to a quadratic term. Actual laboratory data are necessary to test the significance of flexural strength in CRCP performance.

DETERMINATION OF A BASELINE DISTRIBUTION

APPROACH

The first step in fitting a model of the type depicted in Eq 4.9 was to determine a baseline distribution for the failure time variable. The failure rate of pavements can be expected to depend on the previous history of cumulative single wheel load applications (W_{eq}), i.e., it seems reasonable to expect the probability of failure to increase as the pavement becomes more and more worn out by the increasing traffic, which is expressed in terms of W_{eq}. The Weibull distribution features these characteristics. In fact, Eq 4.6 shows that the probability of survival changes exponentially with the failure time variable in a Weibull distribution. In addition, Ref 5 states that the Weibull distribution is usually adequate to model failure time of any system that fails by wear-out of one or more of its components. Since a qualitative analysis indicates the adequacy of the Weibull distribution, the next step consisted of testing the goodness-of-fit of this distribution.

ESTIMATES OF THE SURVIVOR FUNCTION

Equation 4.8 shows the goodness-of-fit test for a Weibull distribution that was applied in this study. A graphical goodness-of-fit test was used, instead of a numerical one, because the graphical method permits a cleared overall view of the departures from the anticipated behavior. The estimates of the survivor function were obtained using the Kaplan-Meier method, which was chosen because it provides non-parametric maximum likelihood estimates, thus eliminating the need for a priori assumptions about the behavior of the variable. In addition, it is capable of handling censored data, a very important feature in this study, where a considerable amount of data are censored. Censored data relates to those locations which have not reached failure. The Kaplan-Meier method is briefly described below, after material from Ref 16.

Let $t_1 < t_2 < \ldots t_j \ldots < t_k$ represent the observed failure times in a sample of size n_0, from a homogeneous population with survivor function S(T). The total number of items at risk at a time immediately prior to t_j is

$$n_j = (m_j + d_j) + \ldots + (m_k + d_k) \qquad (4.11)$$

where

d_j = number of items that fail at time t_j, and
m_j = number of censored items in the interval $[t_j, t_{j+1}]$

and the probability of failure at time t_j is

$$P(T=t_j) = S(t_j) - S(t_j+0) \qquad (4.12)$$

where

$$S(tj+0) = \lim_{x \to 0+} S(tj + x) \qquad (4.13)$$

The contribution to the likelihood of a survival time censored at t_j is assumed to be equal to $S(t_j+0)$ in Eq 4.12, i.e., it is assumed that the only information the observed censored time t_j can provide is that the failure time is greater than t_j. Under these conditions, the likelihood function on the space of survivor functions S(T) is

$$£ = \prod_{j=0}^{k} \left([S(tj) - S(tj + 0)]^{dj} \prod_{l=1}^{mj} S(tjl + 0) \right) \qquad (4.14)$$

where all the parameters are as in Eqs 4.11, 4.12, and 4.13, and t_jl represents the censored survival times. Reference 16 proves that £ is maximized for the following estimate of the survivor function:

$$\hat{S}(T)] = \prod_{j:tj<T} \left(\frac{nj - dj}{nj} \right) \qquad (4.15)$$

Equation 4.15 represents the Kaplan-Meier estimate of the survivor function, also called product-limit estimate by some authors (Refs 5, 9, and 16). It makes the conditional probability of failure at each t_j agree with the corresponding observed conditional relative frequency. Equation 4.15 was applied to the data to find a baseline distribution for the cumulative number of equivalent single axle loads applications.

RESULTS

Figure 4.5 shows the plot of the logarithm of the negative of the logarithm of the survivor estimates obtained using Eq 4.15, versus the logarithm of the cumulative number of equivalent single axle loads applications, the time variable. A sample from a Weibull distribution would produce an approximately linear plot, as shown in Eq 4.8.

Figure 4.5 reveals that, the probability of survival decreases with W_{eq} somewhat slower than exponentially, especially for the sections with heavy traffic. Departures from exponentially are primarily due to a few observations of the cumulative number of equivalent single axle loads applications (W_{eq}), either on the upper or on the lower extremes. If these are removed from the data, the trend becomes linear. Since pavements that carry either very low or very high traffic volumes are not subject to the same design and maintenance procedures normally used in other pavements, the adherence of these extreme cases to a model cannot be anticipated, and their removal from a sample used to predict the average statewide performance is justifiable.

The data show a very slight trend to split into two strata. The significance of this apparent heterogeneity needed further testing, despite the fact that Fig 4.5 suggests its probable non-significance.

The homogeneity was tested using non-parametric rank tests. This type of test was chosen because rank tests are less sensitive to outliers, than the corresponding parametric tests. In addition, they involve only a small loss in efficiency compared to an appropriate parametric procedure (Ref 16). Therefore, homogeneity can be tested over the entire W_{eq} range, without significant loss in efficiency, using non-parametric procedures.

Due to the existence of outliers in the low and high extremes, generalized forms of the Wilcoxon test (Ref 16), which places more weight on early survival times, and of the log-rank test (Ref 16), which places more weight on large survival times, were both applied,

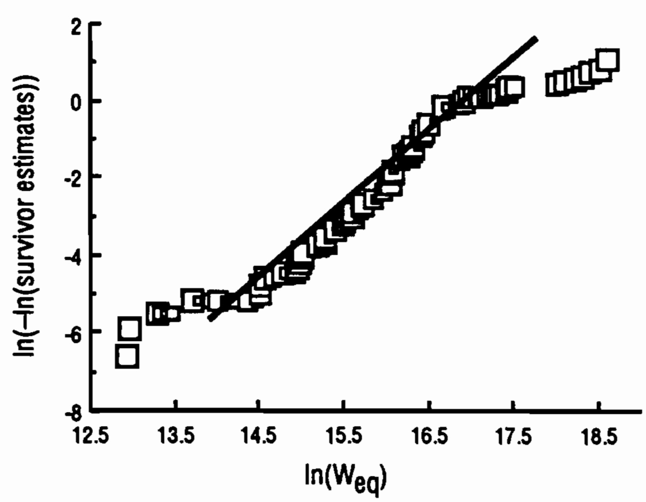


Fig 4.5. Goodness-of-fit test of the Weibull distribution.

using an existent statistical package (Ref 35). The results were:

(1) Log-rank: $\chi_1^2 = 107.8$

(2) Wilcoxon: $\chi_1^2 = 92.2$

The probability level associated with these values of (chi-squared with 1 degree of freedom) is smaller than 0.001. Since neither of the tests was significant, it was concluded that the sample is homogeneous.

The next step was to determine a regression model to predict the distribution of the logarithm of the cumulative number of equivalent single axle loads applications (ln W_{eq}), conditioned to the concomitant variables, based on the assumption that the underlying distribution of the failure time variable (W_{eq}) is Weibull.

CALIBRATION OF THE MODELS

According to the results of the goodness-of-fit test, the Weibull distribution is valid for the following ranges the cumulative number of equivalent single axle loads applications (W_{eq}):

$$14.25 \leq \ln (W_{eq}) \leq 17$$

or

$$2 * 10^6 \leq W_{eq} \leq 2.42 * 10^7 \quad (4.16)$$

The model given by Eq 4.9 was calibrated over the range depicted in Eq 4.16, using an existent statistical package capable of interactively determining the maximum likelihood estimates of the regression parameters (Ref 35). All the candidate variables listed in the beginning of this chapter were individually tested for significance in a preliminary screening. Qualitative variables, such as SOIL or CAT, have their significance levels derived from a composite chi-square statistic for testing whether there is any effect from any of the levels of the variables (Ref 35). Table 4.2 summarizes the significance levels obtained with these individual runs. The significant results are boldface in Table 4.2.

The variable thickness was tested in two different ways as a continuous numerical variable and as a categorical variable, because only three values of thickness (8, 9 and 13 inches) exist in the statewide CRCP data base. The significance level did not increase much from the first to the second test, as shown in Table 4.2.

The variables DEFL (normalized area under the deflection basin) and NDF1 (normalized deflection under the load) were tested to check whether or not the non-significance of the variables back-calculated from deflections was due to the errors of the back-calculation process. The significance levels of the coefficients of DEFL and NDF1 are smaller than those of the back-calculated variables, but they is still high. It is worth of noting, however, that the load transfer coefficient (J) was significant; since it is also back-calculated from deflections, it already reflects the significance of the pavement response to load in the pavement performance.

Three significant models were obtained combining the variables in Table 4.2. Table 4.3 summarizes the results of the calibration process of the models listed below. In model 3, the addition of the load transfer coefficient caused the significance level of the coefficient of thickness to raise to 8.37 percent.

$$(1) \quad \ln (W_{eq})_q = 17.357 - 0.602\,\Delta Z + 0.273\,\omega_q \quad (4.17)$$

$$(2) \quad \ln (W_{eq})_q = -25.000 - 0.581\,\Delta Z + 5.292\,D + 0.250\omega_q \quad (4.18)$$

$$(3) \quad \ln (W_{eq})_q = -25.000 - 0.953\,\Delta Z - 2.186\,J + 0.250\,\omega_q \quad (4.19)$$

where

ln = natural logarithm,
W_{eq} = cumulative number of equivalent single axle load applications,
ΔZ = distress index differential,
D = slab thickness,
J = load transfer coefficient,
subscript q = distribution quantile, and
ω = type 1 asymptote of minima, tabulated in Appendix D.

The algorithm used for calibrating these models starts by obtaining initial estimates of the regression parameters, through ordinary least squares, with both censored and non-censored observations, ignoring the censoring. The log-likelihood function is then maximized using the Newton-Raphson algorithm, an interactive procedure described in detail in Refs 5, 11, and 16. When lack of convergence happens, providing initial estimates

TABLE 4.2. SUMMARY OF SIGNIFICANCE LEVELS OF THE CONCOMITANT VARIABLES

ΔZ	Initial – current distress index	0.10
J	Load transfer coefficient	2.43
E_{SB}	Elasticity modulus of the subbase	8.91
RAIN	Average annual rainfall	9.47
D	Slab thickness (numerical)	10.38
CATD	Slab thickness (categorical)	12.15
DEFL	Normalized area under deflection basin	26.57
k	Modulus of reaction on top of the subbase	30.51
SBT	Subbase type	35.86
E_c	Elasticity modulus of the PC concrete	50.29
TEMP	Average annual lowest temperature	56.17
E_{SG}	Elasticity modulus of the subgrade	67.45
SOIL	Swelling potential	72.80
CAT	Coarse aggregate type of the PC concrete	86.54
C_d	Drainage coefficient	90.64
NDF1	Normalized deflection under the load	98.49

TABLE 4.3. SUMMARY OF RESULTS OF THE CALIBRATION PROCESS					
Model	Parameter	Value	DF	Significance Level	χ^2
1	Intercept	17.357	1	0.0001	36850.800
	Coefficient of ΔZ	-0.602	1	0.0001	49.956
	Scale	0.273	1	–	–
2	Intercept	-25.000	*	–	–
	Coefficient of D	5.292	1	0.0001	253382.000
	Coefficient of ΔZ	-0.581	1	0.0001	53.061
	Scale	0.250	*	–	–
3	Intercept	-25.000	*	–	–
	Coefficient of D	6.194	1	0.0837	2.991
	Coefficient of ΔZ	-0.953	1	0.0001	201.164
	Coefficient of J	-2.186	1	0.0243	5.076
	Scale	0.250	*	–	–

*Fixed parameter

for some parameters may speed up the interactions and improve convergence. Lack of convergence happened in the cases of models 2 and 3. They required that successive runs be made, each of them with the parameters initialized according to the results of the previous runs. When little change in these coefficients was observed, a final run with some fixed parameters was made, which yielded the results depicted in Table 4.3. The fixed parameters have a star (*) in the column corresponding to the number of degrees of freedom.

TABLE 4.4. COMPARISON BETWEEN PREDICTED AND OBSERVED MEDIANS FOR MODEL 1			
Δ Z	Observed Median	Predicted Median	Percent Difference
0.0	15.910	17.257	-8.5
0.5	15.860	16.956	-6.9
1.0	16.381	16.655	-1.67
1.5	16.121	16.354	-1.45
2.0	16.229	16.053	1.08
2.5	15.361	15.752	-2.55

TEST AND INTERPRETATION OF MODEL 1

COMPARISON BETWEEN PREDICTED AND OBSERVED VALUES

Since the models calibrated in this study predict a survival distribution for the dependent variable, comparisons between the predicted and observed medians of ln (W_{eq}) were made. For model 1, the results are summarized in Table 4.4. In the calculation of the medians, ΔZ was divided into classes whose mid-points are represented in Table 4.4, and this mid-point was substituted in the Model 1 equation, to obtain the predicted medians. The percent difference shown in Table 4.4 is calculated as

$$\text{Percent dif} = ([\text{obs} - \text{pred}] / \text{obs}) \, 100 \qquad (4.20)$$

Despite the fact that the calculation of observed ΔZ medians and the use of class mid-points already introduce some error in the process, the predicted and observed values are very close for Model 1, which can be considered reasonably accurate. Figure 4.6 illustrates this result.

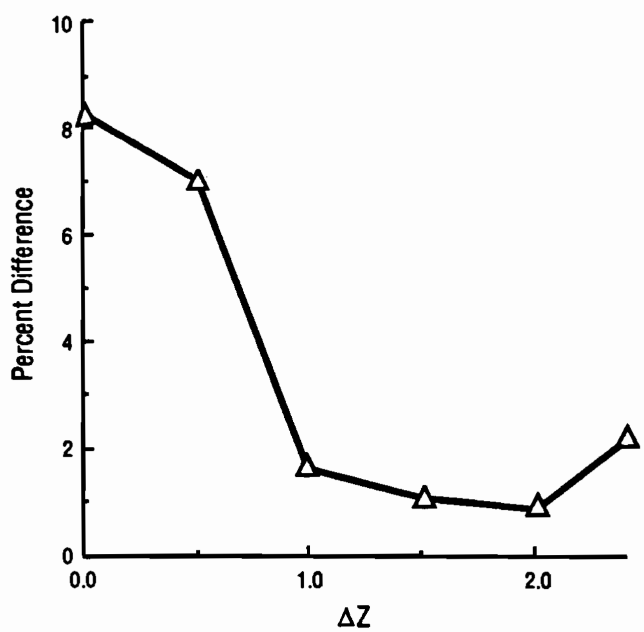


Fig 4.6. Trends of prediction errors with Z-score-model 1.

INTERPRETATION OF THE MODEL

For each level of pavement distress, expressed in terms of ΔZ, model 1 predicts the survivor, or reliability, function conditioned to the given ΔZ. This concept is illustrated in Fig 4.3.

The model 1 equation was calibrated using data from a CRCP network constructed and designed under strict specifications, which have the objective of eliminating the effect of each bad factor affecting performance, so that the pavements perform well. The non-significant coefficients assigned to the variables are the statistical translation of this fact. The mathematical technique used to arrive at model 1 considers a binary variable representing overlaid and non-overlaid pavements, i.e., the occurrence of failure at a given number of traffic repetitions. When a test section is overlaid, i.e., has failed, it has a high value of ΔZ, and the relationship between ΔZ and occurrence of failure is captured by the model. This influence is very clear in the data, as shown in Fig 4.7. Figure 4.8 shows that there is no clear trend of ΔZ with traffic repetitions.

Model 1 predicts the probability of survival of a pavement that is in a particular condition in terms of distress. For example, the model can determine which of two CRC pavement sections, one at the ΔZ_1 level, the other at the ΔZ_2 levels, has the highest probability of surviving a traffic level of $W_{eq}0$, given that all other characteristics are statistically the same. Evidently, if $\Delta Z_1 > \Delta Z_2$ (section 2 is in better condition than section 1), the model predicts a higher probability of survival for section 2. It is important to note that the model cannot reflect the traffic or distress history. In other words, since the available sample precludes consideration of any effect

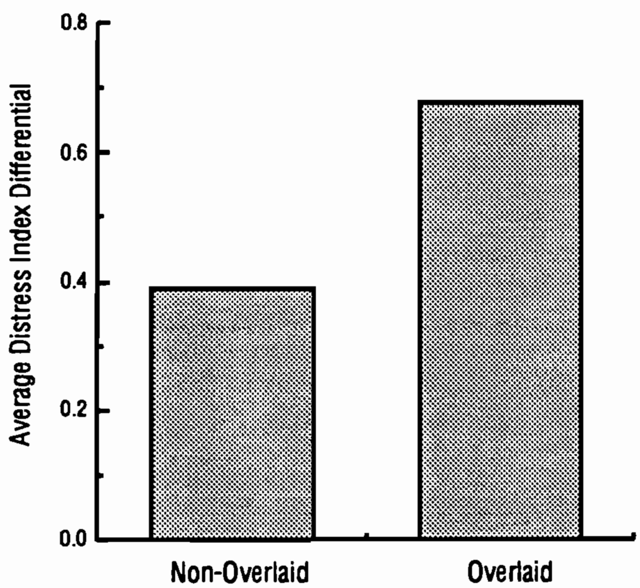


Fig 4.7. Distress index differential and occurrence of failure.

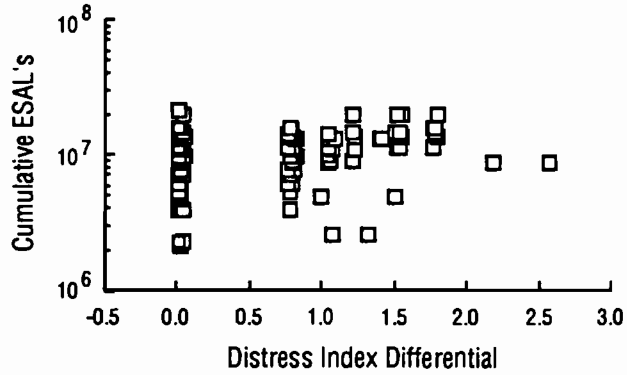


Fig 4.8. Variation of the distress index differential with traffic.

other than distress condition, and since the data show a clear relationship only between ΔZ and occurrence of failure, the model is simply comparing CRCP sections that are identical except for the amount of distress. This is conceptually the opposite of a model that predicts what happens to a pavement that has undergone a certain amount of traffic, which caused a certain amount of distress.

SIGNIFICANCE OF DISTRESS MANIFESTATIONS AS EXPLANATORY VARIABLES

As discussed earlier in this document, the Z-score has some limitations, which are dealt with by means of the already described data reduction process. Although the condition survey data themselves also have limitations, models explaining the performance in terms of three distress manifestations were attempted. These distress manifestations are

(1) number of minor punchouts per mile,
(2) number of severe punchouts per mile, and
(3) number of patches per mile.

These models could provide an additional check of the significance of distress manifestations on CRCP performance. However, no convergence was achieved with models using these types of distress manifestations as explanatory variables. The reasons for this can be found in a detailed examination of the available CRCP condition survey data. The survey procedures changed from survey year to survey year, for reasons that go from budget limitations to the immediate need for data to use in some research study. Consequently, conflicting distress information is found more often than is desirable. For example, there are several cases of distress manifestations disappearing without a correspondent increase in another distress manifestation. These are some typical examples.

(1) Test section 1003-4-W had zero punchouts and one patch recorded in the 1982 survey. In the 1984 survey, all distress manifestations were zero, and the section had not been overlaid.

(2) Test section 4010-2-W had 37 punchouts per mile recorded in the 1974 survey, no distress manifestations recorded in the 1978 survey, 21 punchouts per mile in the 1982 survey, and no distress manifestations in the 1984 survey. In 1987, the section had been overlaid.

These examples are not unusual in the data base; but removal of conflicting data would drastically reduce the available information. The data reduction process applied to the Z-score automatically compensated for this limitation in a logical way, thus enabling a meaningful analysis of CRCP performance.

TEST AND INTERPRETATION OF MODEL 2

COMPARISON BETWEEN PREDICTED AND OBSERVED VALUES

Comparisons between the predicted and observed medians of ln (W_{eq}) were also made for model 2. The class groups for ΔZ were the same as those used in Table 4.4. For thickness, it was not necessary to build classes, because the sample comprises only three levels of thickness: 8, 9 and 13 inches. The percent deviations were calculated according to Eq 4.20. The comparisons are summarized in Table 4.5.

For model 2, the differences between observed and predicted values are very small for a slab thickness of 8 inches and very high for the other two thicknesses available in the CRCP data base. This is illustrated in Fig 4.9. The ranges of validity of the survival distribution, considered in Eq 4.16, imply that Model 2 is valid only for thicknesses of around 8 inches. This is illustrated in Fig 4.10, which also shows a comparison between thickness prediction with Model 2 and with the AASHTO model, at a 50 percent reliability level. The predicted values from the AASHTO model were calculated using the values of the explanatory variables listed below.

$$\Delta PSI = 4.5\text{-}2.5$$
$$Cd = 1$$
$$J = 2.6$$
$$E_c = 5*10^6 \text{ psi}$$
$$f_f = 650 \text{ psi and } 850 \text{ psi}$$
$$k = 460 \text{ pci}$$

The values correspond either to average values available in the CRCP data base or to values recommended in Refs 1 and 2. Since the CRCP data base does not contain conclusive data on the flexural strength of the PC concrete, Fig 4.10 was developed to show the extremes of the band of flexural strengths typically used in Texas.

The data plotted for model 2 in Fig 4.10 show a steep relationship between slab thickness and cumulative equivalent single axle loads. The small range of validity of the data results from the fact that 700 of the data points come from 8-inch-thick CRCP sections, while only 35 come from 9-inch-thick sections and 12 from 13-inch-thick sections. It also results from the fact that only 91 data points, out of a total of 747 data points, had reached a failure condition, and all failed sections have 8-inch-thick slabs. The survival analysis, however, has taken this

TABLE 4.5. COMPARISON BETWEEN PREDICTED AND OBSERVED MEDIANS FOR MODEL 2

D	ΔZ	Number of Observations	Observed Median	Predicted Median	Percent Difference
8	0.0	158	15.91	17.24	-8.38
8	0.1	1	16.58	17.18	-3.63
8	0.7	4	15.86	16.84	-6.16
8	0.8	24	16.03	16.78	-4.66
8	0.9	56	16.29	16.72	-2.64
8	1.0	98	16.45	16.66	-1.28
8	1.1	2	15.69	16.60	-5.84
8	1.2	3	16.02	16.55	-3.29
8	1.3	3	14.77	16.49	-11.66
8	1.4	1	16.38	16.43	-0.26
8	1.5	4	16.22	16.37	-0.93
8	1.8	1	16.49	16.20	1.76
8	2.2	1	15.97	15.96	0.04
8	2.4	1	14.77	15.85	-7.33
8	2.6	1	15.96	15.73	1.41
9	0.0	11	14.46	22.53	-55.84
13	1.0	2	18.61	43.12	-131.62

small amount of censored data (data which has not reached failure) into consideration, and the result suggests strongly that the effect of slab thickness is greater than is presented in the AASHTO model. Figure 4.10 does not imply that the effect of increasing from 8 to 9 inches in thickness would, for example, yield a tenfold increase in traffic as shown in the dashed line, but it does strongly indicate that it would yield more than the two-fold increase in traffic shown by the AASHTO model. While the restrictions in the data preclude conclusive results, there is strong evidence here to suggest that some modification is warranted in the current AASHTO model.

FURTHER TESTS OF SIGNIFICANCE OF THICKNESS

Before conclusions can be drawn from model 2, the meaningfulness of using thickness as an explanatory variable requires further testing, because of the following conflicting facts:

(1) Model 2 is more sensitive to thickness than the AASHTO model, even within its narrow range of validity (Fig 4.10).

(2) Conversations with engineers who have many years of practical experience with Texas CRC pavements indicate that the AASHTO model may not capture the importance of thickness to CRCP performance. This fact agrees with the results of model 2.

(3) The prediction errors with model 2 were extremely high for the 9 and 13-inch-thick slabs. The values of W_{eq} predicted for these cases are outside the range of validity of the model, depicted in Eq 4.16 and in Fig 4.10.

(4) In the CRCP data base, 700 out 747 sections have 8-inch-thick slabs. This makes the experimental data extremely unbalanced. The fact that some significance resulting from thickness could be captured despite this problem confirms the opinions of the expert engineers but does not satisfactorily quantify the effect of thickness.

(5) During the calibration of model 2, non-convergence of the algorithm had to be overcome by fixing two of the model parameters. Lack of convergence may be due to a strong correlation between the two explanatory variables, and it is reasonable to anticipate existence of correlation between distress manifestations and thickness.

The strategy used to verify the actual significance of thickness as an explanatory variable in model 2 considered thickness as a categorical rather than a continuous variable, because the CRCP test sections have only three values of thickness. The first tentative model was calibrated with thickness divided into two categories only:

(1) 8 and 9 inches and

(2) 13 inches.

The resulting model was exactly the same as model 1, and the categorical thickness variable was non-significant. A cross-check of this model was done by redefining the variable that represents the distress index differential according to the three available levels of thickness, as follows:

(1) $\Delta Z8 = \Delta Z$ if thickness = 8 inches, zero otherwise;

(2) $\Delta Z9 = \Delta Z$ if thickness = 9 inches, zero otherwise;

(3) $\Delta Z13 = \Delta Z$ if thickness = 13 inches, zero otherwise.

A model was calibrated using $\Delta Z8$, $\Delta Z9$ and $\Delta Z13$ instead of ΔZ as explanatory variables. Again, the resulting model was exactly the same as model 1 for a thickness level of 8, and non-significant otherwise. It is thus concluded that results from model 2 confirm the expert engineers' opinion that the AASHTO model does not capture the importance of thickness in the performance of Texas CRC pavements, but that the prevalence of a single value of thickness in the CRC pavements statewide precludes a satisfactory quantification of this effect.

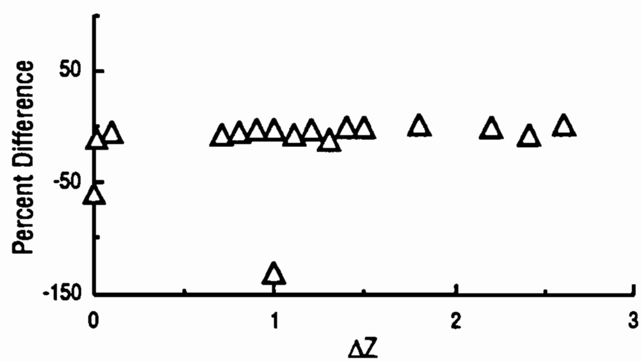


Fig 4.9. Trends of prediction errors with Z-score – model 2.

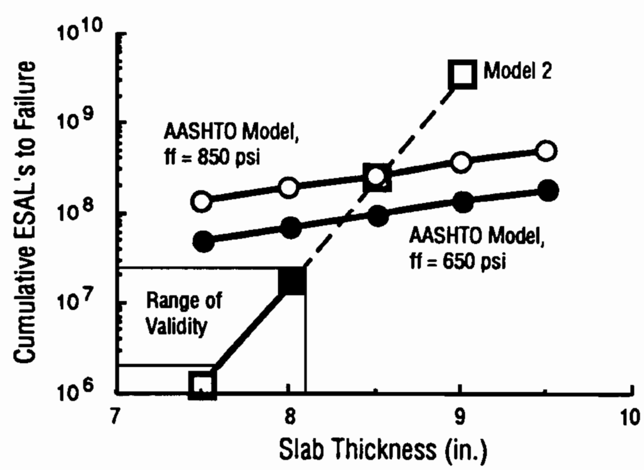


Fig 4.10. Comparison between model 2 and AASHTO model.

CONCLUSIONS AND RECOMMENDATIONS

PRACTICAL USES OF THE MODELS

The main motivation for this study was the fact that the AASHTO model is not a good representation of the failure and performance of Texas CRC pavements. The ideal situation would have been to arrive at another model featuring all variables known to affect performance, one that could be used either to design new pavements or to evaluate existing ones. However, only very restrictive models were obtained, basically because the available sample does not span a wide enough range of the explanatory variables. This is due to the fact that the sample comes from existing pavements, built according to standards and specifications that seek well-performing pavements statewide. It also results from the fact that the data consist basically of non-overlaid 8-inch-thick test sections, as shown in Table 4.1. Despite the limitations inherent in the sample, the results show that it is possible to accurately estimate the remaining life of a CRC pavement using information from the visual condition survey.

Since the currently available J-values consist of preliminary results of ongoing research and rely on a weak assumption, the practical use of model 3 is not recommended at this stage, unless some specific analysis concerning J-value of a CRCP section is needed. The interpretation of the results must take into account the fact that the J-values used to calibrate the model are only preliminary estimates.

RANGE OF OBSERVED INFORMATION

The characteristics of the available data affect the results obtained in this study. These data are summarized in Table 4.1, and their most important limitation is the predominance of non-overlaid, 8-inch-thick pavements. The statistical method chosen to analyze the data has the capability to consider the information provided by censored data, and this permitted a meaningful analysis of the currently available data. However, the only information provided by non-overlaid sections is that the equivalent single axle load applications at failure is greater than the values given by the data. In other words, it is not known at this point how long the non-overlaid test sections will last, which means that the distribution of a considerable portion of the number of equivalent single axle load applications at failure, as well as the format of the distribution is unknown. This idea is illustrated in Fig 4.11, which shows in dashed lines the still unknown distribution, with the portion corresponding to the available failure data highlighted. The survival distribution of the known data is conceptually shown in the smaller plot in Fig 4.11.

It is also important to note that the known data correspond to sections that failed sooner than the unknown mean of the total distribution, and that a Weibull model fitted a considerable portion of the data (Fig 4.5). Since the Weibull distribution is also the type 3 asymptote of minima (Ref 5), the survival distribution fitted to the data is probably reflecting the distribution of the minimum extreme values of the still unknown underlying distribution of the W_{eq} to failure for Texas CRCP. This gives additional information about possible types of distributions for the W_{eq} to failure, because it can be proved that extreme minimum values from underlying log-normal, gamma, beta and Weibull distributions give rise to a type 3 asymptote of minima (Ref 5).

RELIABILITY ESTIMATES

The reliability in the AASHTO model comes from an additive term that transforms the regression estimate of the decimal logarithm of the traffic into the mean of a normal random variable with a standard deviation whose values are recommended in Ref 2. However, "it is acknowledged that the recommended values for the standard deviation have been derived through a series of judgements on previously reported values for components

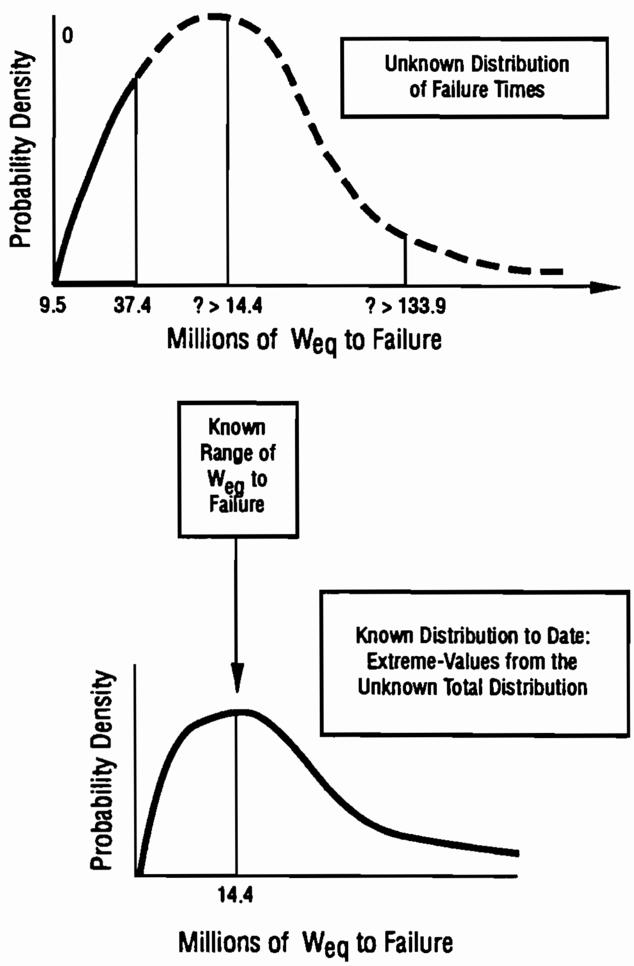


Fig 4.11. Conceptual comparison between the available data and the total failure time distribution.

of the overall variance. Little or no objective data exist for certain components, particularly for variances associated with design factors that have been newly introduced into the revised design equations, e.g., drainage coefficients" (Ref 2).

The use of Survival Analysis technique provides models that yield not a point estimate of the failure time variable but a direct estimate of the survivor (or reliability) function, as depicted in Fig 4.3. This is obtained as a built-in estimate of the probability of failure, obtained directly by calibrating a theoretically sound mathematical model capable of generating the desired survivor (or reliability) function of the variable. This approach is much more sound from a theoretical point of view than that used in the AASHTO model (Refs 1 and 2), where the reliability of a pavement section is estimated through an additive term that consists of the product of the normal deviate and some covariance of design parameters. The basic underlying assumption hidden in the AASHTO's concept of reliability (Refs 1 and 2) is worthy of further discussion, which can be found in Chapter 5.

ADDITIONAL COMMENTS

The non-significance of a series of variables known to affect pavement performance is probably due to the fact that the available sample is restricted to the values that correspond to design practices. While the AASHTO model was fitted using data from a designed experiment with wide ranges of the influencing variables, the models derived in this study reflect the design and construction practices employed in Texas, which reflect a preoccupation with avoiding combinations of design variables known or suspected to cause bad performance. The resulting non-significance of the majority of variables shows that the current design and construction practices are generating good pavements, which have shown to be unaffected by factors such as subgrade swell, rainfall, temperature, and some of its interactions. The prevalence of non-overlaid sections in the CRCP data base is an additional indication of this fact.

The derived models confirmed the finding of the AASHTO model, that pavement performance is sensitive to the logarithm of the cumulative number of equivalent single axle loads.

The models derived in this study have the underlying assumption that ΔZ, distress index differential, and thickness (D) are known with certainty, and the uncertainty in the model comes primarily from the traffic data. Although the assumption of certainty is not correct, it is reasonable to assume that the variations of traffic data are considerably higher than those of pavement thickness. If the distress index (Z-score) did not have the limitations discussed earlier in this document, it would be a very accurate measurement of CRCP deterioration, because data on a number of easily recognizable distress manifestations, such as patches and punchouts, are probably almost error-proof and 100 percent reproducible. Considering all these facts, the approach of the proposed models is justifiable and theoretically sound.

CHAPTER 5. DISCUSSION

THE FRAMEWORK OF PAVEMENT PERFORMANCE STUDIES

The quality of the results of an experiment aiming to analyze any phenomenon in terms of influencing factors depend primarily on the type and amount of data available. It is imperative that the range of values observed for the influencing factors, or independent variables, be large enough to show significance through statistical analysis. This objective can be attained by the design of controlled experiments, in which adequate ranges of values are created for controllable factors, in order to ensure that the statistical methods can detect the influence and give a satisfactory quantitative estimate of the magnitude of the factors.

In the case of pavement performance studies, it is known from theory and experience that the following factors can affect pavement performance:

(1) structural characteristics, e.g., thickness of layers;
(2) properties of component materials, e.g., flexural strength of PC concrete;
(3) environmental factors, e.g., amount of rainfall;
(4) foundation characteristics, e.g., subgrade potential swell;
(5) traffic, i.e., the load; and
(6) pavement condition, e.g., sealed versus non-sealed cracks.

The influence of most of these factors on pavement performance has been recognized, and existing pavements usually reflect design practices to account for the effects of these variables. Good illustrations of these practices can be found in the CRCP data base. For example, cement or asphalt-stabilized subbases are always found in areas subject to high amounts of rainfall and high subgrade potential swell. Conversely, granular subbases are found exclusively in areas where the combined effects of environmental and foundation factors are less harmful to the pavements. Therefore, any pavement network built according to reliable standards does not provide an adequate sample for detecting the influence of concomitant variables on performance, because most pavements are constructed such that many variables are counterbalanced with design factors. Figure 5.1 shows a scheme that compares the range of the desired inference space to the range spanned by standard designs and specifications. In order to guarantee that a statistical model can capture the significant effect of a variable on performance, a wide range of values of the variable must be available in the data, as shown in the three axes of the hypothetical inference space in Fig 5.1. In practice, however, pavements are always designed and built within a small range of this inference space, which is likely to yield satisfactory performance. This subset is also shown in Fig 5.1, where the thickened lines on the axes represent the ranges spanned by the standards and specifications.

Controlled experiments, such as the AASHO Road Test, attempt to overcome this problem. However, in these experiments, only controllable factors, such as thickness, materials, and traffic, can be made representative of a wide range of conditions. The environmental factors reflect only the conditions at the road test site, and extensions of road test results to other conditions require data from pavements under those conditions, which are restricted by design practices, and cannot reflect the ranges usually found in experimental designs.

Using a more exact terminology, the ideal sample should provide data over the entire inference space desired, i.e., data from all possible combinations of controllable and uncontrollable factors. Nevertheless, in real life, the samples always come from a population that represents some subset of the desired sample space, reflecting restrictions imposed either by construction practices or by the climatic conditions or by a combination of these factors. The solution of this dilemma is neither straightforward nor inexpensive. It relies on a very wide-range performance study, which may or may not entirely span the desired inference space, depending on the strictness of the construction and design practices used in the test sections available for survey.

At the current state-of-the-art, the best that can be done is to use data from local samples, which are usually

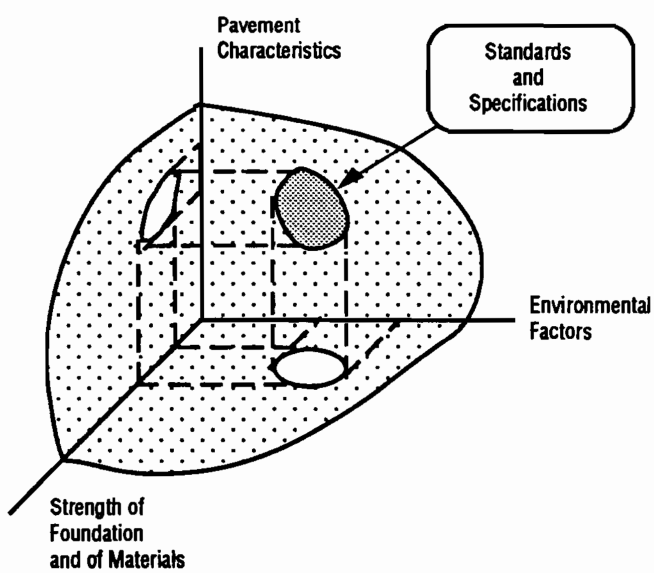


Fig 5.1. Restriction in the inference space for pavement performance studies.

affected by one major type of inference space restriction, in conjunction with results from designed experiments, which are affected by another major type of restriction.

The models derived in this study are representative of the conditions prevalent in the atate of Texas, and they are strictly valid only for pavements built and designed according to current Texas standards and specifications.

THE PROBLEM OF ERROR PROPAGATION

INTRODUCTION

Assuming that a set of test sections spanning the ideal unrestricted inference space is available, another very important aspect of any study concerned with data analysis and empirical modeling still remains unsolved: error propagation.

The ultimate objective of any performance study is to arrive at a model that predicts pavement performance as a function of the design values of the dependent variables. The determination of these values is thus a major step in arriving at a performance prediction model. Even in the hypothetical event that accurate records of construction specifications are easily retrievable, strict adherence to specifications is not easily attained in the field, and the characteristics of pavement sections built under the same specifications are always subject to variation. These problems motivated a discussion of the consequences of the error propagation on the modeling process, using as examples the impact of the following variables on a model of CRCP performance:

(1) elastic modulus of the PC concrete (E_c),
(2) flexural strength of the PC concrete (f_f), and
(3) modulus of reaction on top of the subbase (k).

In the majority of real-life situations, estimates of the initial values of the variables above must be obtained using data from expeditious field surveys. This discussion will assume that design records of the three variables above are unknown and that they will be back-calculated from deflections measured in the field.

Since the true nature of the phenomena being discussed here is still unknown, the discussion will remain qualitative. The mathematical formulas for errors are meant only to illustrate the expected pattern of error propagation and superposition.

NOTATION

The following notation is used throughout this chapter:

(1) *Notation for the error terms:*

ε = errors of completely unknown magnitude, or due to assumptions of dubious validity;

ξ = errors due to some known approximation, estimate, or measurement process; and

λ = errors known to be undesirably high.

(2) *Notation for the subscripts of the error terms:*

Subscript E refers to E_c (elasticity modulus of PC concrete);

Subscript k refers to k (modulus of reaction on top of subbase); and

Subscript f refers to f_f (flexural strength of PC concrete).

Additional subscripts are used to designate the particular source of error for each of the variables. Propagation and superposition of errors are represented by a bracketed plus sign [+], to indicate that the errors propagate in some unknown fashion, which may or may not be additive.

ERRORS DUE TO USE OF VALUES FOR A GIVEN SET OF CONDITIONS AS SURROGATES FOR THE DESIGN VALUES

(1) *For the elasticity modulus of the PC concrete (E_c)*

For a given PC concrete, E_c is a function of moisture (m) and time (t). Increase of E_c with time can be estimated because the section age is known. As for the influence of moisture, it is usually impossible to estimate correctly, because the practical constraints of a statewide survey do not permit moisture content tests to be performed. The error is thus

$$\varepsilon_E = \varepsilon_{Em} [+] \xi_{Et} \tag{5.1}$$

(2) *For the flexural strength of the PC concrete (f_f)*

The considerations for E_c apply to f_f. Thus, the error is

$$\varepsilon_f = \varepsilon_{fm} [+] \xi_{ft} \tag{5.2}$$

(3) *For the modulus of reaction on top of subbase (k)*

The modulus of reaction on top of subbase varies with season (s), with moisture contents of the subgrade (m), and with pavement age, or time (t). The error introduced by the use of any particular k-value as a surrogate for the design value can be expressed as

$$\varepsilon_k = \varepsilon_{ks} [+] \varepsilon_{kt} [+] \varepsilon_{km} \tag{5.3}$$

ERRORS DUE TO BACK-CALCULATING PARAMETERS FROM DEFLECTIONS

The basic sources of these errors have already been discussed in Chapter 3. They are listed here in order to help the visualization of the overall error, which will affect all estimates of E_c, f_f, and k. The overall errors due to the back-calculation process for E_c, f_f, and k, respectively, are

$$\varepsilon_E = \lambda_B [+] \xi_{FWD} \tag{5.4}$$

$$\varepsilon_f = \lambda_B \, [+] \, \xi_{FWD} \tag{5.5}$$

and

$$\varepsilon_k = \lambda_B \, [+] \, \xi_{FWD} \tag{5.6}$$

where

λ_B = error due to failures in the underlying assumptions of the back-calculation process (e.g., axi-symmetry of the structure) combined with error inherent in the back-calculation process; and

ξ_{FWD} = error intrinsic to the equipment used to measure deflections, the falling weight deflectometer (FWD) in this study.

For the case of deflections due to load at the interior of the slab, the error due to the unaccounted-for moisture effect and the error due to the unaccounted-for temperature effect can both be considered negligible, according to experimental evidence (Ref 28).

ERRORS PROPAGATED IN A MODEL WITH E_c, f_f, AND k AS DEPENDENT VARIABLES

When back-calculated E_c, f_f, and k are used to fit a statistical model to any performance variable, values of the independent variable predicted with the model are affected by the following combination of propagated errors:

$$\begin{aligned}\varepsilon_{pred} = {} & \varepsilon_{Em} \, [+] \, \xi_{Ect} \, [+] \, \varepsilon_{fm} \, [+] \, \xi_{ft} \\ & [+] \, \varepsilon_{ks} \, [+] \, \varepsilon_{kt} \, [+] \, \varepsilon_{km} \\ & [+] \, \lambda_B \, [+] \, \xi_{FWD} \\ & [+] \, \lambda_B \, [+] \, \xi_{FWD} \\ & [+] \, \lambda_B \, [+] \, \xi_{FWD} \\ & [+] \, \psi \end{aligned} \tag{5.7}$$

where

ε_{pred} = error in the prediction of the independent variable;

ψ = error due to other sources;

other errors = as described previously.

Although the magnitudes of all these errors are unknown, at least three of them are known to be undesirably high (λ). In order to check whether or not these errors were causing non-significance of these variables in the proposed models obtained in this study, an alternate model subject to considerably less error was tried during the calibration process.

TENTATIVE COURSE OF ACTION

In Table 4.2, it can be seen that two surrogates for the design values of E_c, f_f, and k, were tried. One was the normalized area under the deflection basin, and the other was the normalized deflection under the load. The reasoning is as follows:

(1) If a regressor variable X is an error-free function of A, the regression of the dependent variable on A will be as good an explanation of the phenomenon as the regression on X.

(2) The only components of error in a model with deflection basins as surrogates for the design values of f_f, E_c and k are

$$\varepsilon_{pred} = \xi_{FWD} \, [+] \, \varepsilon_m \tag{5.8}$$

i.e., the error intrinsic to the equipment used for taking deflection basins measurements (x_{FWD}) and the error due to the effects of unknown moisture gradients in the strength of the test sections (ε_m).

The substitution of back-calculated design variables for the normalized area under the deflection basins drastically decreases the expected errors and the significance levels of the coefficients in the model. Despite this reduction, these coefficients (Table 4.2) are still non-significant. This probably indicates the uniformity of foundation and of the PC concrete, due to design and construction practices, as the most likely cause of non-significance of variables in the CRCP model.

CONCLUSIONS AND ADDITIONAL COMMENTS

The brief analysis made above for three model inputs can be extended to other variables, such as load transfer coefficients and Z-score. Further research work in the modeling of CRCP performance should always keep in mind the substantial increase in the expected error that addition of a new variable can bring to a model. It is believed that more accurate conclusions about the nature of the CRCP performance can be drawn with models that carefully avoid use of estimated surrogate variables affected by propagation of errors.

It seems opportune to remark that one of the most important variables affecting pavement performance – the traffic – is always subject to a considerable amount of unavoidable, uncontrollable error. Accurate traffic data can be obtained only if traffic counters and weigh stations are installed in each and every test section. In real life, this is infeasible, and traffic data consist of extrapolations from relatively few counters and weigh stations scattered over the network. Whether or not the costs of improving upon accuracy of traffic data are worth the benefits is a question beyond the scope of this study. The important point is that any model encompassing traffic data will always carry in it the considerable errors intrinsic to these data; introduction in the model of variables subject to similar or even greater errors may render any model useless for both practical and research purposes.

RELIABILITY OF AN ENGINEERING DESIGN

DEFINITION

The behavior of most engineering structures is complex, and they are subject to a set of service conditions which are also complex. These conditions often deviate from their anticipated values, they are difficult to measure or quantify, and their effects on the life of the structure interact in a complex way. Life-length is a random variable, and its actual value depends on the design and construction, or manufacturing, practices and on the quality of the materials. Hence, reliability, or probability of survival, is conditioned upon a stated set of conditions affecting the structure. Statistical reliability information is provided by the distribution model of the life-length random variable (Ref 5) and is usually represented by the survivor function, or reliability function, as explained in Chapter 4. There are three basic approaches to modeling the reliability of a system that consists of several components.

(1) A physical model of the failure mechanism can be constructed on the basis of theoretical knowledge about this mechanism, and, in a second stage, the variability of the component's characteristics is incorporated into the model. This is the most accurate approach, but only when the nature of the failure mechanism is well known.

(2) A model of the system life-length can be derived directly from observations of the in-service structure. This approach is less accurate than the case above, but it is the best when the nature of the failure mechanism is not known, because it uses actual performance data.

(3) A model for the system life-length can be derived from information on component reliability. For the cases of engineering devices such as circuits, where information on the component reliability can be obtained either from the manufacturer or testing in the laboratory, this approach is the most convenient (Ref 5).

Since the nature of the failure mechanism and the concept of pavement performance are very complex and still controversial, approach 1 can be applied only if some existing empirical model is assumed to be an acceptable physical explanation of pavement performance and failure. The drawbacks of such an assumption are self-explanatory in the case of pavements, perhaps the most complex of all engineering structures. Approach number 3 is inconvenient, because the pavement component's reliability is ill-defined, if defined at all. For example, some arbitrary measurement of life-length of asphaltic concrete can be determined in the laboratory using cores, but the measurement may or may not make sense for an asphaltic concrete base layer. It is easier, cheaper and more accurate to measure the life-length of the in-service pavements. Consequently, it follows that approach 2 is convenient and sound for CRC pavements, or pavements in general.

RELIABILITY ESTIMATES WITH THE AASHTO MODEL AND WITH THE PROPOSED MODEL

The AASHTO model is an empirical model partially calibrated with data from a designed experiment. Its invaluable contribution resides in the fact that the data used for calibration spanned a very wide range of the inference space, thus providing a model that suffers relatively small limitations of the type described in Fig 5.1. This model was calibrated using least squares, and reliability was introduced in a second phase. For rigid pavements, the model is

$$\log(W_{18}) = Z\alpha\, S_O + 7.35 \log(D+1) - 0.06 + \frac{\log\left[\dfrac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \dfrac{1.624 \times 10^7}{(D = 1)^{8.46}}}$$

$$(4.22 - 0.32\, p_t) \log\left[\frac{f_f\, C_d\, [D^{0.75} - 1.132]}{215.63\, J \left[D^{0.75} - \dfrac{18.42}{\left[\dfrac{E_c}{k}\right]^{0.25}}\right]}\right] \tag{5.1}$$

where

W_{18} = cumulative equivalent 18-kip single axle load applications,

Z_a = standard normal deviate, for a probability a,

S_0 = combined standard error of the traffic prediction and performance predictions,

ΔPSI = initial – final PSI,

p_t = terminal PSI,

D = slab thickness (inches),

f_f = flexural strength of PC concrete (psi),

J = load transfer coefficient,

C_d = drainage coefficient,

E_c = modulus of elasticity of PC concrete (psi),

k = modulus of subgrade reaction (pci), and

logs = base 10.

In this model, the part that predicts the mean value of the pavement was originally obtained applying least squares regression techniques to the data, and the reliability term was introduced later (Ref 2). Conceptually, this model is

$$\log(W_{18}) = Z_\alpha\, S_O + \overline{\log(W_{18})} \tag{5.2}$$

where

$\overline{\log(W_{18})}$ = life-length estimate obtained with Eq 5.1, and the other terms are as in Eq 5.1.

In Eq 5.2, the left-hand side is a normal random variable of variance S_O^2 and mean $\overline{\log(W_{18})}$. In other words, the concept of reliability embedded in the AASHTO model is

> For any value of the explanatory variables of the model (Eq 5.1), the predicted value $\overline{\log(W_{18})}$ is the true mean of a normal random variable with a variance S_O^2, which is constant and independent of the values of the explanatory variables.

An analysis of the mathematical meaning of the term $\overline{\log(W_{18})}$, the point estimate from the AASHTO model, indicates that the concept of reliability used in the derivation of Eq 5.1 is not correct. In fact, the quantity $\overline{\log(W_{18})}$ is not a true value but an estimate subject to error. Therefore, it has a variance. For the simplest case, that of linear regression of a dependent variable y on one explanatory variable x, the variance of the estimate of the dependent variable is

$$V(\overline{y_k}) = \frac{\sigma^2}{n} + \frac{\sigma^2(\chi_k-\overline{\chi})^2}{\sum_{j=1}^{n}(\chi_i-\overline{\chi})^2} \quad (5.3)$$

where

$V(\overline{y_k})$ = variance of the estimate of y at x_k using the regression equation,

$\overline{y_k}$ = estimate of y at x_k using the regression equation,

σ^2 = true variance of y, whose estimate is the residual mean squares from regression,

x_k = value of the explanatory variable substituted in the regression equation,

x_i = i^{th} value of the explanatory variable in the sample, and

n = sample size used to obtain the model.

According to Eq 5.3, the standard deviation of the dependent variable estimate (S_o in Eq 5.1) is not constant, and any estimates of this variance that do not take into account the values of the explanatory variables in the sample used to calibrate the model have no mathematical meaning. It is also worth noting that $V(\overline{y_k})$, the variance of dependent variable estimate, increases as the value of the independent variable approaches the extremes of the sample used to obtain the model. Consequently, even if correct estimates of the variance of $\overline{\log(W_{18})}$ at each level of the dependent variables were available, they could be used only within the range of independent variables spanned by the original AASHTO experiment. For any design near the extremes of this range, the true value of the quantity S_o is very high, and the reliability estimate is considerably different than any value obtained by substituting the some rough estimate of S_o in Eq 5.1.

In this study, a model of the system life-length was derived directly from observations of the in-service structure, or device. This is the second basic approach discussed in the beginning of this item. Actual performance data were available for a considerable number of test sections and, despite the sample limitations, a significant model could be obtained using a theoretically sound statistical approach.

CONCLUSION

The success of any study that attempts to quantify the effect of explanatory factors on one or more random variables depends on three key factors:

(1) availability of an adequate sample,
(2) minimization of the errors in the variables and of their propagation into the models, and
(3) correct application and interpretation of the statistical method chosen to analyze the data.

For pavement performance studies, the first and the second factors above are difficult, if not virtually impossible, to attain. In real life, the samples for pavement performance studies always come from a population that reflects restrictions imposed either by construction practices, or by the climatic conditions, or by a combination of these factors. In addition, the data are usually difficult to collect and even to quantify, and the data can be affected by a considerable amount of virtually unavoidable error.

Only the third factor, the correct application of an adequate statistical method, is controllable in pavement performance studies, but a critical examination of the literature in this area reveals that this factor is overlooked more frequently than would be desirable. This research project pioneered the application of survival analysis in pavement performance studies, and the fact that several useful conclusions can be drawn from very restricted data shows the importance of theoretically sound application and interpretation statistical methods in a pavement performance study.

CHAPTER 6. SUMMARY OF FINDINGS AND FINAL RECOMMENDATIONS

SUMMARY OF FINDINGS

PERFORMANCE OF CRC PAVEMENTS

The main objective of this research was to study the performance of continuously reinforced concrete pavements (CRCP) using an existing 14-year CRCP data base, which was expanded with data collected under this study. Despite the restrictions of the data available for this study, Survival Analysis yielded three significant models that can provide estimates of the reliability function of a CRCP, conditioned to the following explanatory variables:

Model 1 = Observed distress index differential;
Model 2 = Slab thickness and observed distress index differential; and
Model 3 = Load transfer coefficient and observed distress index differential.

Although the range of validity of these models is too restricted to permit their wide practical application, the following conclusions can be derived from the results:

(1) The available data on the number of equivalent single axle loads (W_{eq}) follow a Weibull distribution for the interval between 2 and 24.2 million W_{eq} applications. Since the Weibull distribution is also the type 3 asymptote of minima (Ref 5), the distribution fitted to the data probably reflects the distribution of the minimum extreme values of the still unknown underlying distribution of the W_{eq} to failure for Texas CRCP.
(2) Survival probability (or reliability) estimates with model 1 are in good agreement with observed relative frequencies, despite the limitations affecting the Z-score and the restrictions in the available sample.
(3) Model 2 does not satisfactorily quantify the effect of thickness, but it indicates that the AASHTO model may not capture the actual importance of the influence of thickness on CRCP performance. This finding agrees with practical experience with CRCP in Texas.
(4) The non-significance of the effects of several variables in the CRCP performance shows that the current design and construction practices are generating good pavements. The prevalence of non-overlaid sections in the CRCP data base is an additional indication of good performance.
(5) While the restrictions in the data preclude a satisfactory quantification of effects of variables on CRCP performance, there is strong evidence to suggest that some modification is warranted in the AASHTO model, and this clearly points out the need for continuous evaluation of the CRCP test sections.

CRCP DATA BASE

An existing 14-year CRCP data base was used by and expanded in this study. The following findings resulted from the work done to add new information to the CRCP data base:

(1) A very accurate model for predicting equivalent single axle load applications in terms of average daily traffic, percent trucks, average ten heaviest wheel loads, and slab thickness was obtained and used in this study (Chapter 3). The explanatory variables of this models are relatively easy to obtain from the SDHPT records, as opposed to accurate calculations of equivalent single axle load applications.
(2) Values of modulus of reaction on top of subbase (k) were back-calculated from deflections. The values obtained with plate theory were in good agreement with those found in the literature, while k-values obtained with layered theory were four to six times larger than those found in the literature.
(3) A non-parametric test was run to check the influence of some factors on the k-values. The results showed that plate theory k-values can capture some influence of the interactions between rainfall and subgrade swell for high swell in the wettest areas, and low swell in the driest areas. Layered theory k-values did not show any influence from these parameters.

FINAL SUGGESTIONS AND RECOMMENDATIONS

These are the most important suggestions and recommendations derived from this study.

(1) The monitoring of CRCP test sections in Texas should be continued on a periodic basis.
(2) Before the next set of deflection measurements is taken, an experiment should be conducted to determine a schedule that will of reflect seasonal variations. These data should be used to verify the influence of seasonal variations of foundation support on CRCP performance.
(3) The current values of load transfer coefficient (J) were significant in model 3, although based on the assumption that the influence of the temperature gradient on the edge and corner deflections is negligible. It is recommended that some procedure to correct edge deflection measurements for slab temperature gradient be developed. Next, it is suggested that new values of load transfer coefficients be calculated using deflection measurements corrected for temperature gradient. These J-values should be used in further updating of the CRCP model.

(4) The available values of the drainage coefficients are also based on weak assumptions, as discussed in Chapter 3. Their lack of significance in the CRCP model may be due either to this fact, or to construction practices that provide reasonable drainage conditions in any CRC pavement. It is suggested that a careful review of construction practices and design standards regarding all factors affecting drainage be carried out to determine if further work on development of other drainage coefficients is likely to lead to significant coefficients in a performance model.

(5) The problem of error propagation in variables serving as surrogates for design values could be almost entirely avoided if accurate records of project specifications and construction tests for quality control were kept in an appropriate data base. It is suggested that such a data base be developed and that records from every new pavement or overlay be kept for future studies.

(6) As more overlays are placed over CRCP and over already existent overlays, a larger sample enabling the study of overlay performance may be obtained. At this time, survival models analogous to these developed in this study should be developed for the overlays, to provide more reliable estimates of survival probability of an overlay over CRCP. In addition, the conclusions about the significance of variables reflecting the materials and the environment can provide guidance about the adequacy of the overlay design and construction practices reflected by the sample.

According to Refs 3, 6, and 14, it is safe to say that the expenditures in the highway sector in the United States represent the largest amount spent in transportation, and at least $10 billion are spent annually on pavements. Therefore, any improvement in managing this investment can result in considerable savings (Ref 14). This latter fact, which, except for the budget figures, also holds for any other country, has been motivating worldwide attempts to better manage the roadway network. A crucial part of this overall research effort consists of understanding and modeling pavement performance, and this study is a part of this effort.

It is felt that the most important contributions of this study are the theoretically sound definition of the problem of reliability estimates for pavements and the pioneer application of survival analysis for solving it. The conceptual discussions about the consequences of the uncertainty of the variables and about the restrictions in the inference space are also valuable guidance for future studies intended to analyze and quantify the pavement deterioration.

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APPENDIX A. PROGRAMS USED IN THE BACK-CALCULATION OF REACTION MODULI

```
/*****************************************************
              PROGRAM TO REDUCE DEFLECTION DATA
*****************************************************/

OPTIONS REPLACE;
CMS FI SDS DISK DUMMY DUMMY Q;
CMS FI SDX DISK DUMMY DUMMY A;
CMS FI SDZ DISK DUMMY DUMMY B;
/*--------------------------------
              MAKE A DATA SET WITH HEIGHT=4 AND
              GOOD INTERIOR DEFLECTION BASINS ONLY
-------------------------------------------*/
DATA A; RETAIN NORME NORMI ;SET SDS.FWD;
IF OVR='Y'; IF HEIGHT=4;
/*--------------------------------
              1.LINEARLY INTERPOLATE MISSING VALUES
              WHEN THERE IS ONLY ONE IN A BASIN
-------------------------------------------*/
IF DF7=. THEN DF7=0.0001;
IF DF6=. THEN DF6=(DF7+DF5)/2;
IF DF5=. THEN DF5=(DF6+DF4)/2;
IF DF4=. THEN DF4=(DF5+DF3)/2;
IF CONF='C' THEN DO;
IF DF3=. THEN DF3=(DF4+DF2)/2;
IF DF2=. THEN DF2=(DF3+DF1)/2;
IF DF1=. THEN DF1=1.2*DF2;   END;
IF CONF='A' THEN DO;
IF DF3=. THEN DF3=(DF1+DF4)/2;
IF DF2=. THEN DF2=DF3;
IF DF1=. THEN DF1=1.2*(DF2+DF3/2);   END;
/*--------------------------------
        2. DELETE DEFLECTION BASINS THAT DEPART TOO MUCH
              FROM EXPECTED PATTERN
-------------------------------------------*/
IF CONF='C' THEN DO;
  IF ((DF1-DF2+(.05*DF1))<0) OR
    ((DF2-DF3+(.05*DF2))<0) OR
    ((DF3-DF4+(.05*DF3))<0) OR
    ((DF4-DF5+(.05*DF4))<0) OR
```

```
    ((DF5-DF6+(.05*DF5))<0) OR
    ((DF6-DF7+(.05*DF6))<0)  THEN DELETE; END;
IF CONF='A' THEN DO;
  IF ((DF1-DF3+(.05*DF1))<0) OR
    ((DF3-DF4+(.05*DF3))<0) OR
    ((DF4-DF5+(.05*DF4))<0) OR
    ((DF5-DF6+(.05*DF5))<0) OR
    ((DF6-DF7+(.05*DF6))<0) THEN DELETE; END;
/*-------------------------------------------
        3. DELETE SUBSECTIONS THAT HAVE INTERIOR(2) RESPONSE WORSE
                THAN THAT AT THE EDGE(1)
-------------------------------------------*/
IF STATION=1 THEN NORME=DF1/LBS;
IF STATION=2 THEN NORMI=DF1/LBS;
IF (NORMI<NORME) AND (STATION=2) THEN OUTPUT;
KEEP CFTR SECT DIR STATION SS LBS DF1-DF7 CONF STEMP;

PROC SORT DATA=A OUT=SDZ.FDEFL;BY CFTR SECT DIR SS;
TITLE 'DATA SET SDZ.FDEFL';
PROC PRINT;                RUN;

DATA SDZ.FPEDEF; MERGE SDZ.FDEFL SDX.CUMOVTC(IN=OK2);  BY CFTR;
IF  OK2;
PROC SORT; BY CFTR SECT DIR SS;
TITLE 'DATA SET SDZ.FPEDEF - FOR FPEDD';
PROC PRINT;                      RUN;


/*******************************************************
   PROGRAM TO CHOOSE A REPRESENTATIVE BASIN PER SECTION
*******************************************************/
OPTIONS REPLACE;
CMS FI SDZ DISK DUMMY DUMMY B;
DATA A; SET SDZ.FPED;

NDF1=DF1/LBS;
NDF2=DF2/LBS;
NDF3=DF3/LBS;
NDF4=DF4/LBS;
NDF5=DF5/LBS;
NDF6=DF6/LBS;
```

```
NDF7=DF7/LBS;
IF CONF='C' THEN DO;
  DEFL=(NDF1+2*NDF2+2*NDF3+2*NDF4+2*NDF5+2*NDF6+NDF7)*6;
              END;
IF CONF='A' THEN DO;
  DEFL=(NDF2+2*NDF1+2*NDF3+2*NDF4+2*NDF5+2*NDF6+NDF7)*6;
              END;
PROC SORT; BY CFTR SECT DIR SS;
/*********** FIND OUT HOW MANY OBS PER BY GROUP ******/
DATA OV1; SET A; BY CFTR SECT DIR SS;
IF FIRST.DIR THEN COUNT=_N_;   RETAIN COUNT;
COUNT2=_N_; IF  LAST.DIR THEN
COUNT3=COUNT2-COUNT+1;  OUTPUT OV1;

DATA AUXOV; SET OV1; BY CFTR SECT DIR SS; IF LAST.DIR;COUNT4=COUNT3;
KEEP CFTR SECT DIR COUNT4;
DATA OV2; MERGE OV1 AUXOV; BY CFTR SECT DIR;  DROP COUNT COUNT2 COUNT3;
PROC SORT DATA=OV2; BY CFTR SECT DIR DEFL;
/************* MEDIAN BASIN ******************************/
DATA OV3; SET OV2; BY CFTR SECT DIR DEFL;
IF COUNT4=10 THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+5; RETAIN XXX ;   END;
IF COUNT4=9  THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+4; RETAIN XXX ;   END;
IF COUNT4=8  THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+4; RETAIN XXX ;   END;
IF COUNT4=7  THEN  DO;
     IF FIRST.DIR  THEN
     XXX= _N_+3; RETAIN XXX ;   END;
IF COUNT4=6  THEN  DO;
     IF FIRST.DIR  THEN
     XXX= _N_+3; RETAIN XXX ;  END;
IF COUNT4=5  THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+2; RETAIN XXX ;   END;
IF COUNT4=4  THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+2; RETAIN XXX ;   END;
```

```
IF COUNT4=3  THEN DO;
   IF FIRST.DIR  THEN
     XXX=_N_+1; RETAIN XXX ;   END;
IF COUNT4=2  THEN  DO;
     IF FIRST.DIR  THEN
     XXX= _N_+1; RETAIN XXX ;
     END;
IF COUNT4=1  THEN
     XXX=_N_  ; RETAIN XXX ;

/** MAKE ONLY ONE DATA SET WITH DEFLECTIONS CORRESPONDING TO THE
      MEDIAN NORMALIZED VARIABLE DEFL ************************/
DATA SDZ.FPERR; SET OV3;
IF _N_=XXX;
PROC SORT DATA=SDZ.FPERR OUT=SDZ.FPERR;
BY CFTR SECT DIR;

/*******************************************************
       PROGRAM  TO  WRITE  INPUT  FILE  FOR  FPEDD1
*******************************************************/
CMS FI SDZ DISK DUMMY DUMMY B;
CMS FI SDS DISK DUMMY DUMMY E;
CMS FI OUT DISK FPED INPUT B (LRECL 100 BLKSIZE 100;
DATA A; MERGE SDS.MASTER  SDZ.FPED2(IN=OK);
BY CFTR; IF OK;
KEEP CFTR SECT DIR SS  LBS DF1-DF7 STEMP OVTC   CONF
   D CAT SBT SOIL;
DATA B; SET A;  FILE OUT LRECL=100;
SBT1=2; IF SBT=4 THEN SBT1=1;
/*1*/     PUT @ 5 '1';
/*2*/     PUT @ 1 '999'
            @ 5 '$' @ 6 CFTR @ 12 SECT @ 14 DIR @ 16 SS @ 18 CONF
            @ 21 STEMP;
/*3*/     PUT @ 1 'NO INFO' @ 22 'FWD';
/*4*/     PUT @ 5 '2' @;
IF CONF='C' THEN
        PUT @ 10 '7' LBS 11-18 @ 19 '.' @ 31 '5.9';
ELSE      PUT @ 10 '6' LBS 11-18 @ 19 '.' @ 31 '5.9';
/*5*/     PUT @ 5 '0'
            @ 10 '0'
```

```
         @ 15 '0'
         @ 20 '2' @;
IF SBT=4 THEN
        PUT @ 25 '1' @;
ELSE       PUT @ 25 '2' @;
        PUT @ 26 '120.1'
/*------NOTE: ALL OVERLAID CRCP ARE IN GOOD CONDITION. ---*/
         @ 40 '0'
         @ 45 '1'; /*-----SKIP REMAINING LIFE ANALYSIS------*/

/*6*/ IF CONF='C' THEN
        PUT @ 1  DF1 @ 11 DF2 @ 21 DF3 @ 31 DF4
         @ 41 DF5 @ 51 DF6 @ 61 DF7 ;
IF CONF='A' THEN
        PUT @ 1 DF1 @ 11 DF3 @ 21 DF4 @ 31 DF5 @ 41 DF6 @ 51 DF7;
IF NOT(STEMP=.) THEN
/*7*/     PUT @ 5 '4' @ 6 STEMP @ 16 '80' @ 18 '.' @;
ELSE       PUT @ 5 '4' @ 6 '105.' @ 16 '80' @ 18 '.' @;
     /* DESIGN TEMPERATURE IS ASSUMED 80 F */
IF CONF='C' THEN
        PUT @ 36 '12.' @ 41 '24.' @ 46 '36.' @ 51 '48.' @ 56 '60.'
         @ 61 '72.' ;
ELSE
        PUT @ 36 '12.' @ 41 '24.' @ 46 '36.' @ 51 '48.'
         @ 56 '60.';
/*--------CARD FOR THE OVERLAY--------------------------*/
/*8.1*/   PUT @ 10 '1' @ 11 OVTC  @ 21 '.30' @ 31 '300000.'
         @ 41 '500000.' @ 51 '100000.';
/*--------------CARD FOR THE CRCP LAYER------------------*/
/*8.2*/   PUT @ 10 '2' D 11-17 @ 18 '.' @ 21 '.15' @ 31 '0.' @;
IF CAT=1 THEN
        PUT @ 31 '5000000.' @ 41 '5800000.'  @ 51 '5100000.';
IF CAT=2 THEN
        PUT @ 31 '5800000.' @ 41 '6700000.'  @ 51 '5000000.';
IF CAT=3 THEN
        PUT @ 31 '5350000.' @ 41 '5700000.'  @ 51 '5000000.';
IF CAT=4 THEN
        PUT @ 31 '4500000.' @ 41 '6000000.'  @ 51 '3500000.';
IF CAT=5 THEN
        PUT @ 31 '4500000.' @ 41 '6000000.'  @ 51 '3500000.';
```

```
/*-----------CARD FOR THE SUBBASE-----------------------*/
/*8.3*/   PUT @ 10 '3' @;
/*------THICKNESS=4 FOR ASPHALT TREATED, ELSE 6--------------*/
IF SBT=1 THEN
        PUT @ 17 '4' @ 18 '.' @;
ELSE
        PUT @ 17 '6' @ 18 '.' @;
IF SBT=4 THEN
        PUT @ 21 '.40' @ 31 '60000.' @ 41 '90000.' @ 51 '30000.';
ELSE
        PUT @ 21 '.32' @ 31 '200000.' @ 41 '300000.' @ 51 '90000.';
/*----------CARD FOR THE SUBGRADE---------------------*/
/*8.4*/   PUT @ 10 '4' @ 21 '.45'  @;
IF SOIL='H' THEN PUT @ 31 '16000.' @ 41 '30000.' @ 51 '7500.';
IF SOIL='L' THEN PUT @ 31 '35000.' @ 41 '55000.' @ 51 '15000.';
/*9*/     PUT @ 4 '20';
/*10*/    PUT;

/********************************************************
PROGRAM TO CREATE DATA SET WITH  SECTIONS WHERE PREVIOUS
             RUN TURNED OUT TO HAVE AN ERROR >= 35%
********************************************************/
OPTIONS REPLACE;
CMS FI SDZ DISK DUMMY DUMMY B;
CMS FI SDS DISK DUMMY DUMMY E;
CMS FI IN DISK HIGH ERROR B;

DATA FIX; INFILE IN; INPUT CFTR SECT DIR $  SS $;
PROC SORT; BY CFTR SECT DIR SS;
DATA FIX1;  SET FIX; BY CFTR SECT DIR; IF FIRST.DIR;
DATA FIX2; SET FIX; BY CFTR SECT DIR; IF LAST.DIR;
DATA FIX3; SET FIX; BY CFTR SECT DIR;
IF ((NOT LAST.DIR) OR (NOT FIRST.DIR));
DATA A; MERGE FIX1(IN=OK) SDZ.FPED; BY CFTR SECT DIR;IF OK;
PROC SORT; BY CFTR SECT DIR SS;
DATA B; MERGE A FIX1(IN=OK); BY CFTR SECT DIR SS; IF (NOT OK);
DATA C; MERGE B FIX2(IN=OK); BY CFTR SECT DIR SS; IF (NOT OK);
DATA D; MERGE C FIX3(IN=OK); BY CFTR SECT DIR SS; IF (NOT OK);
DATA E; MERGE SDS.MASTER   D(IN=OK);
BY CFTR; IF OK;
```

```
KEEP CFTR SECT DIR SS DF1-DF7 CONF STEMP LBS OVTC
    D CAT SBT SOIL;
PROC SORT OUT=SDZ.HIERR; BY CFTR SECT DIR SS;
PROC PRINT;


/******************************************************
       PROGRAM TO CALCULATE K-VALUES WITH AASHTO EQUATION
******************************************************/
OPTIONS REPLACE;
CMS FI SDZ DISK DUMMY DUMMY B;
CMS FI SDS DISK DUMMY DUMMY E;
CMS FI IN DISK KNOV OUT A;

DATA A; MERGE SDS.MASTER   SDZ.OVES(IN=OK);
BY CFTR; IF OK;
IF SBT=1 THEN DSB= 4; ELSE DSB=6;
KEEP CFTR SECT DIR  DSB D E1-E4;

DATA OV;      SET A;
LGK=-2.807+0.1253*((LOG(DSB))**2)+1.062*LOG(E4)
   +0.1282*(LOG(DSB))*(LOG(E3))-0.4114*(LOG(DSB))
   -0.0581*(LOG(E3))-0.1317*(LOG(DSB))*(LOG(E4));
K=ROUND(EXP(LGK));
OVR='Y';
KEEP CFTR SECT DIR K OVR;
```

APPENDIX B. RESULTS OF THE BACK-CALCULATION PROCESS

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
1001	1	W	289	5800	200	23	34.879	1415	–
1001	2	W	286	5800	200	26	34.999	1533	–
1001	3	W	274	5800	200	24	34.476	1472	–
1001	4	W	298	5800	200	24	33.926	1477	–
1001	5	W	342	5800	200	21	29.233	1322	–
1001	6	W	293	5800	200	26	34.498	1556	–
1003	1	W	288	5800	118	26	22.014	1430	–
1003	2	W	221	5800	200	30	27.788	1751	–
1003	3	W	455	5800	200	16	20.977	1042	–
1003	4	W	500	5800	200	16	16.809	1042	–
1003	5	W	500	5800	200	16	26.175	1042	–
1003	6	W	496	5800	200	22	34.219	1391	–
1005	2	W	290	5000	138	29	23.377	1621	–
1005	3	W	179	5000	90	32	31.293	1630	–
1005	6	E	292	5000	131	30	30.608	1648	–
1015	1	E	–	5000	90	25	8.53	1380	261
1015	2	E	–	522	90	23	7.01	2296	473
1015	3	W	–	5000	90	43	9.62	2047	442
1015	4	W	–	5657	90	23	7.01	1496	289
1015	5	W	–	5000	90	27	5.44	1231	272
2002	1	E	–	4900	90	26	7.3	1364	311
2002	2	E	–	4900	90	29	7.32	1484	534
2002	3	E	–	5667	113	25	4.52	1765	490
2002	4	E	–	4900	90	46	15.05	2169	350
2002	5	E	–	4900	90	45	16.99	1481	425

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
2002	6	W	–	–	–	–	–	–	2500
2028	1	N	–	5903	30	28	6.72	1184	335
2028	1	S	–	5360	53	44	3.73	1845	606
2028	2	N	–	5019	30	47	10.89	1294	390
2031	1	E	284	5450	154	24	33.415	1373	–
2031	1	W	230	5100	131	47	30.714	2347	–
2031	2	E	361	5100	144	23	24.063	1371	–
2031	2	W	443	5100	150	22	14.578	1314	–
2031	3	W	264	5100	200	47	34.958	2539	–
2031	4	W	393	5450	200	31	34.992	1819	–
2032	1	E	–	5549	34	31	4.43	1370	427
2032	1	W	–	5413	62	70	70.47	–	1746
2032	2	W	–	5000	30	55	13.97	2096	379
2032	2	W	–	5000	30	55	13.97	–	662
2032	3	E	–	5000	30	31	11.68	1315	311
2041	1	N	–	5726	144	27	2.87	1538	342
2041	1	S	–	5950	90	23	2.41	1620	345
2041	2	N	–	6700	171	27	8.33	1083	352
2044	1	N	243	5800	200	33	74.63	–	–
2044	1	S	217	5800	200	55	89.542	–	–
2044	2	N	244	5800	200	32	74.438	–	–
2044	2	S	–	5000	90	43	12.86	1380	730
2044	3	S	–	5850	280	70	109.67	–	–
2044	4	S	–	5000	90	63	19.5	3275	1000
2044	5	S	–	5620	194	42	2.85	1934	584

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
2046	1	N	500	5450	200	35	24.582	1854	–
2046	1	S	500	5450	200	35	24.981	1854	–
2046	2	N	500	5450	200	35	15.704	1854	–
2049	1	N	–	5000	90	70	21.45	3100	901
2049	1	S	–	5000	100	60	14.33	2880	735
2049	2	S	–	6495	90	50	6.25	2128	630
2049	3	S	–	5000	90	31	18.81	2296	486
2049	4	S	–	5085	280	70	48.65	–	1236
2050	1	N	–	5000	90	34	16.99	1645	557
2050	1	S	–	6245	90	22	6.32	1145	271
2050	2	S	–	6137	90	26	9.87	1071	215
2051	1	E	–	5347	90	31	9.99	1814	500
2051	1	W	–	6229	90	29	7.23	1458	369
2051	2	E	–	5293	90	28	5.13	1523	380
2059	1	E	–	6584	165	32	7.73	2235	513
2059	1	W	–	5000	90	30	6.63	1342	371
2059	2	E	–	5000	90	30	7.72	1274	301
2059	2	W	–	5000	90	40	15.78	1870	396
2060	1	E	–	5998	90	21	7.46	1318	286
2060	1	W	–	5000	90	44	6.88	1507	379
2060	2	W	–	6642	259	34	5.53	1592	398
2075	1	N	–	6700	181	53	6.08	1442	533
2075	1	S	–	5719	300	42	4.72	1693	526
2075	2	S	–	6107	132	42	4.98	2061	544
2075	3	S	–	6151	124	37	3.88	3295	998

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
2075	4	S	–	5000	90	38	20.12	1797	1694
2098	1	E	–	5000	117	45	7.76	2189	627
2098	1	W	–	5000	92	70	22.18	3109	871
2098	2	E	–	5850	280	70	117.94	–	1576
2098	2	W	–	5000	169	65	7.74	2490	488
3001	1	N	–	5000	90	30	9.22	3319	367
3001	2	N	–	5000	90	28	10.71	1653	316
3010	1	S	–	5000	90	22	6.71	1369	248
3010	2	S	–	5679	90	18	7.29	1025	193
3010	3	S	–	–	–	–	–	–	245
3018	1	S	–	5000	90	36	9.95	1503	349
3018	2	S	–	5664	90	34	8.24	1795	436
4002	2	W	426	5450	30	20	17.762	931	–
4009	5	W	–	5500	90	13	4.29	809	188
4010	1	W	207	5450	200	19	28.365	1109	
4010	2	W	231	5450	200	21	33.564	1192	–
4010	3	W	283	5450	200	28	34.734	1556	–
4011	1	E	–	5800	90	16	4.31	823	139
4011	1	W	–	5500	90	23	2.16	1292	256
4011	2	E	–	5500	90	19	4.89	972	141
4011	2	W	–	5500	90	14	6.27	841	144
4011	3	W	–	5500	90	21	3.91	1167	260
4022	1	E	–	5500	90	26	5.26	1148	272
4022	1	W	–	5800	100	19	6.09	1185	199
4022	2	W	–	5500	99	24	4.05	1383	287

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
4025	1	W	–	5800	153	68	6.66	2949	551
4025	2	W	–	5200	97	44	9.73	2728	717
4025	3	W	–	5500	90	19	6.7	577	139
4025	4	W	–	5200	90	23	7.3	1177	299
5005	1	N	–	5800	87	43	7.27	2112	591
5005	1	S	–	5775	87	35	3.4	1856	472
5005	2	N	–	5800	63	39	5.27	1974	520
5005	2	S	–	5800	87	58	4.33	2655	750
5007	1	S	–	5468	90	40	4.25	2005	476
5007	2	S	–	6461	222	43	2.56	3580	821
5007	3	S	–	6700	184	40	4.12	2065	483
5008	1	N	–	5500	139	32	3.11	2032	443
5008	1	S	–	5500	117	29	6.25	1400	349
5008	2	N	–	5800	227	30	5.34	2091	502
5008	2	S	–	5800	264	29	10.27	1518	354
5009	1	N	–	5800	151	33	3.64	1851	402
5009	1	S	–	5800	179	31	7.52	1957	396
5009	2	S	–	5800	141	27	7.41	1964	425
12901	1	E	–	5800	300	56	24.67	3184	738
12901	2	E	–	5800	300	63	10.21	2975	658
12901	3	W	–	5800	300	54	6.79	2830	499
12901	4	W	–	5800	300	54	13.65	3069	580
12902	1	E	–	5800	300	56	18.13	3475	729
12902	2	W	–	5800	300	60	14.21	3608	639
13013	2	W	–	5800	300	32	6.38	2352	563

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
13013	3	W	–	5800	192	55	7.49	2652	642
13013	4	W	–	5800	125	25	7.17	1917	476
13013	5	W	–	5800	252	43	5.64	2199	674
13015	1	W	–	6447	90	27	5	1358	371
13015	2	W	–	6211	101	18	4.32	862	139
13015	3	W	–	6700	139	23	3.64	2123	349
13015	4	E	–	6700	112	23	4.76	1295	204
13015	5	W	–	6577	103	22	3.46	1476	312
15032	2	N	500	5800	200	31	15.465	1811	–
15036	1	N	500	6700	300	30	510.594	–	–
15036	2	N	500	6700	300	30	353.658	–	–
15036	3	N	500	6700	300	30	34.615	1878	–
15901	1	N	–	5850	280	70	55.22	–	600
17003	1	N	390	5450	200	21	24.66	1229	–
17003	2	N	266	5800	200	20	34.839	1179	–
17003	3	N	358	5800	200	20	24.767	1148	–
17003	4	N	290	5450	200	23	32.554	1329	–
17004	1	S	133	5450	200	29	31.124	1573	–
17004	2	S	260	5450	200	16	24.028	931	–
17004	3	S	179	5450	200	25	22.019	1395	–
17004	4	S	196	5450	200	30	33.531	1619	–
17004	5	S	–	5200	90	24	2.86	–	189
17004	5	S	–	5200	90	24	2.86	–	277
17004	5	S	332	5450	200	25	30.84	1276	–
17004	5	S	332	5450	200	25	30.84	–	–

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
17004	5	S	332	5450	200	25	30.84	–	–
17004	6	S	205	5450	200	25	34.053	1386	–
17007	1	S	–	5200	108	30	7.43	1258	309
17007	2	S	–	5500	90	22	3.35	1133	258
17007	3	S	–	5437	118	25	2.15	1449	320
17007	4	S	–	5200	148	31	2.41	1496	346
17007	6	S	–	5569	90	13	1.32	1293	224
17011	1	S	–	5500	90	18	3.63	1292	240
17011	2	S	–	5800	109	15	3.22	1068	229
17011	3	S	–	5200	96	27	3.14	764	174
17011	4	S	–	5200	90	25	5.31	1058	199
17011	5	S	–	5500	90	21	3.19	1037	222
17011	6	S	–	5776	90	18	2.93	1152	198
19001	3	W	305	5100	131	55	10.00	2624	–
19001	4	W	207	5450	90	34	34.699	1678	–
19001	5	W	312	5100	135	55	10.00	2634	–
19001	6	W	500	5450	200	35	34.076	1854	–
19006	1	W	366	5450	200	30	27.046	1751	–
19006	2	W	500	5800	200	30	29.226	1751	–
19006	3	W	500	5800	200	30	34.984	1751	–
19006	4	W	500	5450	154	23	12.376	1387	–
19006	5	W	500	5800	300	30	14.122	1878	–
19010	1	W	425	5450	278	30	26.162	1853	–
19010	2	W	470	5450	300	30	34.074	1878	–
19010	3	W	500	5800	200	30	25.156	1751	–

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
19010	4	W	500	5800	200	30	63.245	–	–
19010	5	W	366	5161	200	30	23.833	1751	–
19010	6	W	262	5100	200	30	34.939	1751	–
19019	1	W	498	5800	200	30	13.037	1751	–
19019	2	W	441	5800	200	273	19.011	1621	–
19019	3	W	500	5800	200	30	26.706	1751	–
19019	4	W	500	5800	200	30	24.845	1751	–
19019	5	W	500	5800	300	30	34.201	1878	–
19019	6	W	457	5800	200	30	17.37	1751	–
20003	1	W	–	5484	99	17	3.03	996	188
20003	2	W	–	5200	90	18	3.92	1065	205
20003	3	E	–	5800	295	19	3.75	1332	244
20003	4	W	–	5800	300	21	6.52	1422	329
20003	5	E	–	5800	143	17	4.14	1268	253
20003	6	W	–	5800	300	18	6.94	1373	213
20009	1	W	287	5800	170	33	20.076	1850	–
20009	2	W	375	5000	200	23	23.077	1424	–
20009	3	W	353	5000	171	25	24.826	1470	–
20009	4	W	368	6700	273	23	26.239	1532	–
20009	5	W	264	6700	300	21	23.223	1447	–
20023	1	W	–	5800	101	18	3.28	930	184
20023	2	W	–	5200	90	20	8.22	1129	195
20023	4	E	–	5200	109	18	3.47	986	192
24006	1	W	–	6690	135	19	3.32	2093	634
24006	2	W	–	5000	90	41	10.94	1536	539

Section ID			Elasticity Moduli (1,000 psi)					Reaction Moduli (pci)	
CFTR	S	D	Overlay	PC Concrete	Subbase	Subgrade	Error	Layered Theory	Plate Theory
24007	1	W	–	–	–	–	–	–	232
24009	1	E	432	5000	200	55	80.914	–	–
24009	3	W	500	5000	128	48	27.202	2352	–
24010	1	W	500	5800	200	42	17.237	2339	–
24010	2	W	500	5800	162	55	14.809	2790	–
24010	3	W	455	5000	200	47	27.141	2578	–
24010	5	E	304	5800	200	50	70.49	–	–
24010	6	W	500	5000	200	54	29.247	2883	–
24014	1	E	140	5000	200	22	34.089	1378	–
24014	2	E	278	5000	200	33	20.933	1906	–
24014	3	E	338	5800	200	55	55.364	–	–
24014	4	E	367	5000	200	55	20.236	2890	–

APPENDIX C. SAMPLE OF THE DATA USED TO CALIBRATE THE MODELS

TABLE C.1. TRAFFIC, GEOMETRIC CHARACTERISTICS, AND CONDITION SURVEY DATA

Project Number	Section	Direction	Year	Equivalent Single Axle Loads	Overlaid?	Date of 1st Overlay	ΔZ	Thickness (in.)	Grading Type
1001	1	W	87	15681174	Y	86	1.0	8	Cut
1001	1	W	84	15681174	N	86	0.9	8	Cut
1001	1	W	82	13541115	N	86	0.0	8	Cut
1001	1	W	80	11183240	N	86	0.0	8	Cut
1001	1	W	78	8876176	N	86	0.0	8	Cut
1001	1	W	74	5265509	N	86	–	8	Cut
1001	2	W	87	15681174	Y	86	1.0	8	Transition
1001	2	W	84	15681174	N	86	0.9	8	Transition
1001	2	W	82	13541115	N	86	0.0	8	Transition
1001	2	W	80	11183240	N	86	0.0	8	Transition
1001	2	W	78	8876176	N	86	0.0	8	Transition
1001	2	W	74	5265509	N	86	–	8	Transition
1001	3	W	87	15681174	Y	86	1.0	8	At grade
1001	3	W	84	15681174	N	86	0.9	8	At grade
1001	3	W	82	13541115	N	86	0.0	8	At grade
1001	3	W	80	11183240	N	86	0.7	8	At grade
1001	3	W	78	8876176	N	86	0.0	8	At grade
1001	3	W	74	5265509	N	86	–	8	At grade
1001	4	W	87	15681174	Y	86	1.0	8	Cut
1001	4	W	84	15681174	N	86	0.9	8	Cut

TABLE C.2. DEFLECTION, ENVIRONMENT, AND MATERIALS

Project Number	Section	Direction	Year	Area Under Basin	Deflection Under Load (mils)	Load of FWD (lbs)	Aggregate Type	Potential Swell	Annual Rainfall (in.)	Temperature (°F)
1001	1	W	87	0.0199	11.28	16176	LS	High	30.7	43
1001	1	W	84	0.0199	11.28	16176	LS	High	30.7	43
1001	1	W	82	0.0199	11.28	16176	LS	High	30.7	43
1001	1	W	80	0.0199	11.28	16176	LS	High	30.7	43
1001	1	W	78	0.0199	11.28	16176	LS	High	30.7	43
1001	1	W	74	0.0199	11.28	16176	LS	High	30.7	43
1001	LS2	W	87	0.0239	12.21	16032	LS	High	30.7	43
1001	LS2	W	84	0.0239	12.21	16032	LS	High	30.7	43
1001	LS2	W	82	0.0239	12.21	16032	LS	High	30.7	43
1001	LS2	W	80	0.0239	12.21	16032	LS	High	30.7	43
1001	LS2	W	78	0.0239	12.21	16032	LS	High	30.7	43
1001	LS2	W	74	0.0239	12.21	16032	LS	High	30.7	43
1001	3	W	87	0.0247	10.36	15656	LS	High	30.7	43
1001	3	W	84	0.0247	10.36	15656	LS	High	30.7	43
1001	3	W	82	0.0247	10.36	15656	LS	High	30.7	43
1001	3	W	80	0.0247	10.36	15656	LS	High	30.7	43
1001	3	W	78	0.0247	10.36	15656	LS	High	30.7	43
1001	3	W	74	0.0247	10.36	15656	LS	High	30.7	43
1001	4	W	87	0.0236	9.52	15800	LS	High	30.7	43
1001	4	W	84	0.0236	9.52	15800	LS	High	30.7	43

TABLE C.3. BACK-CALCULATED DATA

Project Number	Section	Direction	Year	Elastic Moduli, 1,000 psi: Overlay	PC Concrete	Subbase	Subgrade	Reaction Moduli (pci): Layered Theory	Plate Theory
1001	1	W	87	288	5800	200	23	1415	
1001	1	W	84	288	5800	200	23	1415	
1001	1	W	82	288	5800	200	23	1415	
1001	1	W	80	288	5800	200	23	1415	
1001	1	W	78	288	5800	200	23	1415	
1001	1	W	74	288	5800	200	23	1415	
1001	2	W	87	285	5800	200	25	1533	
1001	2	W	84	285	5800	200	25	1533	
1001	2	W	82	285	5800	200	25	1533	
1001	2	W	80	285	5800	200	25	1533	
1001	2	W	78	285	5800	200	25	1533	
1001	2	W	74	285	5800	200	25	1533	
1001	3	W	87	274	5800	200	24	1472	
1001	3	W	84	274	5800	200	24	1472	
1001	3	W	82	274	5800	200	24	1472	
1001	3	W	80	274	5800	200	24	1472	
1001	3	W	78	274	5800	200	24	1472	
1001	3	W	74	274	5800	200	24	1472	
1001	4	W	87	297	5800	200	24	1477	
1001	4	W	84	297	5800	200	24	1477	

APPENDIX D. QUANTILES OF THE STANDARDIZED TYPE 1 ASYMPTOTE OF MINIMA

Survival Probability	Quartile
0.99	-4.600
0.95	-2.970
0.90	-2.250
0.85	-1.817
0.80	-1.500
0.75	-1.280
0.70	-1.031
0.60	-0.672
0.50	-0.367
0.40	-0.087
0.30	0.186
0.20	0.476
0.10	0.834
0.05	1.097