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16. Abstract

This report presents the development and application of a manual for the design of both rigid and flexible overlays for rigid pavements. An important feature of the manual is that it presents approximate hand solutions to typical overlay design projects in the State of Texas.

This report also presents some additions to the Texas Rigid Pavement Overlay Design System. These include procedures to define design inputs and the development of charts for the structural design of overlays. Condition survey data were used to develop a technique for estimating the remaining life of an existing rigid pavement. The computer program RPRDS-1 was used to develop tables for the selection of optimal overlay strategies. Finally, a set of design charts was developed for approximate hand solutions for the design of overlay thickness.

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IMPLEMENTATION OF A COMPREHENSIVE RIGID PAVEMENT OVERLAY DESIGN SYSTEM INTO A CONDENSED OVERLAY DESIGN MANUAL

by

Adrianus W. Viljoen
B. Frank McCullough

Research Report 388-4

Condition Surveys and Performance Monitoring of Existing
and Overlaid Rigid Pavements
Research Project 3-8-84-388

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

May 1985

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 reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report was completed at The University of Texas at Austin Center for Transportation Research, under Project 3-8-84-388, as part of the Cooperative Research Program between The University of Texas and the Texas State Department of Highways and Public Transportation. An objective of this project is to validate the overall design method developed in Project 249. This report presents the results of work done in this regard, in the form of a condensed overlay design manual.

Special acknowledgement is made to the staff of the Center for Transportation Research of The University of Texas at Austin, in particular to Lyn Gabbert for typing the drafts of this report. Special thanks are expressed to Dr. Muthu, Waheed Uddin and Victor Torres-Verdin for their valuable comments and suggestions.

Adrianus W. Viljoen

B. Frank McCullough

LIST OF REPORTS

Report No. 388-1, "Development of a Deflection Distress Index for Project-Level Evaluation of CRC Pavements," by Victor Torres-Verdin and B. Frank McCullough, presents the derivation of a new approach for project-level evaluation of CRC pavements from condition survey data. The main features of computer program DDI1, which incorporates the principal findings from the study, are discussed and an input guide for that program is provided along with a project-level condition survey manual.

Report No. 388-2, "Evaluation of the Effect of Survey Speed on Network-Level Collection of Rigid-Pavement Distress Data," by Victor Torres-Verdin, Chhote Saraf and B. Frank McCullough, describes work done in relation with an experiment performed to evaluate the effect of monitoring speed on the quality of rigid-pavement distress data collected at the network level.

Report No. 388-3, "Manual for Condition Survey of Continuously Reinforced Concrete Pavements and Jointed Concrete Pavements," by Chhote Saraf, Victor Torres-Verdin and B. Frank McCullough, presents the procedures for condition survey of CRC and JC pavements recommended for the Rigid Pavement Evaluation System.

Report No. 388-4, "Implementation of a Comprehensive Rigid Pavement Overlay Design System into a Condensed Overlay Design Manual," by Adrianus W. Viljoen and B. Frank McCullough, presents the development and application of a manual for the design of both rigid and flexible overlays for rigid pavements.

ABSTRACT

This report presents the development and application of a manual for the design of both rigid and flexible overlays for rigid pavements. An important feature of the manual is that it presents approximate hand solutions to typical overlay design projects in the State of Texas.

This report also presents some additions to the Texas Rigid Pavement Overlay Design System. These include procedures to define design inputs and the development of charts for the structural design of overlays. Condition survey data was used to develop a technique for estimating the remaining life of an existing rigid pavement. The computer program RPRDS-1 was used to develop tables for the selection of optimal overlay strategies. Finally, a set of design charts was developed for approximate hand solutions for the design of overlay thickness.

KEYWORDS: Rigid pavements, overlays, remaining life, design strategies, design charts, design manual.

SUMMARY

A Rigid Pavement Overlay Design Manual has been compiled. This manual can be used to obtain approximate hand solutions to overlay design problems, as well as for the preparation of design input to more precise automated design models. The background and development of procedures incorporated in the manual are described in the main body of the report, and the design manual is presented in Appendix A.

The design manual can be broadly classified into four phases. The first phase deals with the collection and reduction of design information. The second phase describes how to use design information to determine design inputs. The design charts and design philosophy for overlay thickness designs are described in the third phase. The last phase shows a simplified procedure to do a net present worth of cost analysis. Finally, a design example is provided.

Significant additions to the Texas Rigid Pavement Overlay Design Procedure are presented in phases 2 and 3. These include: (a) nomograph to predict remaining life from condition survey data. This estimate of remaining life is then compared with the remaining life of the fatigue model and techniques on how to use this comparison to improve the inputs to the design procedure are provided; (b) a set of tables to determine the optimal design strategies for different traffic and subgrade support conditions. These tables can also be used to get an indication of the consequences (in terms of relative costs) when the optimal strategy is not implemented. Finally an estimate of user delay cost as a percentage of total cost is in the tables; and (c) four design charts for overlay thickness design. The charts can be used to predict the pavement response and to determine the design life of typical overlay design strategies.

IMPLEMENTATION STATEMENT

The Texas Rigid Pavement Overlay Design System has been implemented into a condensed design manual. Techniques for the definition of design inputs and set of design charts for approximate hand solutions to typical overlay design projects are provided.

It is recommended that the design charts be used for preliminary design purposes and that the computer programs which form part of the Rigid Pavement Rehabilitation Design System be used for the final design. The techniques and procedures to define design inputs presented in the manual can, however, also be used for the determination of design inputs to more precise automated solutions.

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CHAPTER 4. REHABILITATION DESIGN STRATEGIES

CHAPTER 5. DEVELOPMENT OF DESIGN CHARTS

CHAPTER 6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

CHAPTER 1. INTRODUCTION

BACKGROUND

The rigid pavement overlay design manual developed in this study form part of a series of reports to finalize a project entitled "Condition Surveys and Performance Monitoring of Existing and Overlaid Rigid Pavements," and is based mainly on research done under the preceding project entitled "Implementation of Rigid Pavement Overlay and Design System." These projects are part of a Cooperative Research Program between the State Department of Highways and Public Transportation (SDHPT) and the Center for Transportation Research (CTR) at The University of Texas. Figure 1.1 presents a schematic outline of the overall rigid pavement design system and illustrates how all the segments and design techniques fit together to form a powerful design system. This study deals exclusively with overlay designs at the project level.

As part of these projects, the original Texas Rigid Pavement Overlay Design Procedure (RPOD2) (Ref 1) was implemented on a number of overlay projects. Follow up studies were conducted to evaluate the structural performances of these overlaid sections. The overall research program resulted in various studies, ranging from improvements to the material characterization and fatigue life prediction methods (Ref 2), to the development of sophisticated design systems. Two very important products of this research program are

(1) the Rigid Pavement Network Rehabilitation Scheduling Computer Program (RPR 1) (Ref 3), which prioritizes a set of rigid pavements for rehabilitation within a given time period, and
(2) a comprehensive rigid pavement overlay design systems which incorporates the most recent design and analytical models into a computer program, RPRDS1 (Ref 4).

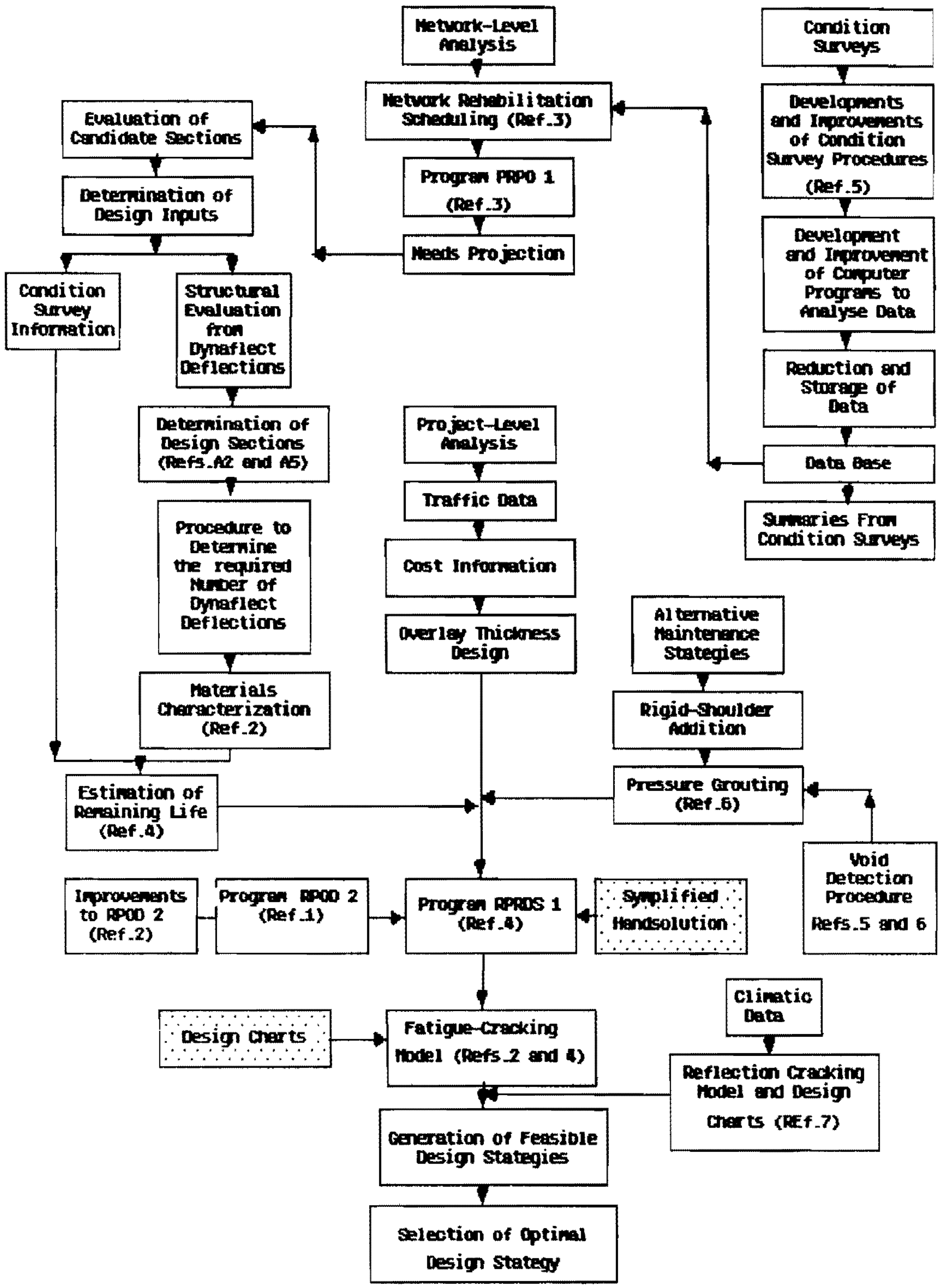


Fig 1.1. The Rigid Pavement Overlay design system and related reports (references to research reports are enclosed in parentheses).

Other important developments were improved pavement evaluation techniques (Refs 5 and 6) and a reflection cracking design model (Ref 7). Due to the large number of factors and factor interactions that affect the design and performance of an overlay, any design procedure that attempts to consider them will necessarily appear complex. The design system is, however, the result of a combination of a number of simple, logical steps. The interaction of these steps and the available design tools is difficult to perceive at first sight. It is necessary to understand the basis of these steps as well as the basic design philosophy to effectively use the available design techniques and computer programs. It was, therefore, considered appropriate toward the end of this project to summarize all the major research findings and design tools in the form of a condensed design manual to guide the designer in the use of the overlay design procedure.

OBJECTIVES

The general goal of this study, then, is to incorporate the major research findings and design tools developed for overlay design at project level into a condensed overlay design manual for easy reference. The specific objectives for achieving this goal are as follows:

(1) to combine research results and field experience to arrive at improved techniques to define important design inputs;
(2) to develop design tables and charts to facilitate the design process;
(3) to incorporate the results of this study and previous studies into a step-by-step design procedure that will introduce the designer to all the important design techniques and models, and provide approximate hand solutions to overlay design projects.

SCOPE

This report includes:

(1) a discussion of data collection for overlay design (Chapter 2),
(2) the development of techniques to facilitate the definition of design inputs (Chapters 3 and 4),
(3) a description of overlay thickness design and the development of design charts (Chapter 5),
(4) a summary of the results of this study and recommendations for further research (Chapter 6).

The overlay design manual developed as part of this study is presented in Appendix A.

CHAPTER 2. DATA COLLECTION AND REDUCTION

Good information about a project is the key to successful and optimal overlay design since it forms the basis of design inputs. A lack of information can, for example, lead to very conservative assumptions that will result in less cost effective designs. The design engineer is often dealing with incomplete information and has to combine information from various sources to arrive at representative design inputs. The objectives of this chapter are threefold:

(1) to identify the basic sources of design information at the project level;
(2) to discuss the data collection procedures;
(3) to present techniques for the basic reduction of data for easy use in the definition of design inputs.

SOURCES OF INFORMATION

The seven basic sources of information that can be used in overlay design are listed in Table 2.1 together with the application of the information in overlay design. It is often necessary to combine a number of these sources to arrive at representative design inputs, as illustrated in Table 2.1. Techniques to combine the various sources of information in the definitions of design inputs are discussed in more detail in Chapter 4.

CONDITION SURVEY

Even though condition survey information is seldom used directly in the overlay design procedure, the results of the condition survey are reflected indirectly in many important design inputs. Information from the condition survey is seldom used alone, but, when combined with other sources of

TABLE 2.1. SOURCES AND APPLICATION OF DESIGN INFORMATION

SOURCE OF DESIGN INFORMATION	APPLICATION IN OVERLAY DESIGN
Condition survey	Design sections; rate of defect development; remaining life; design strategy; maintenance and repair strategy; void detection
Deflection data	Design section; material properties; remaining life; maintenance and repair strategies; load transfer at discontinuities; void detection
Traffic data	Design traffic; remaining life; users delay
Laboratory tests	Material properties; stress sensitivity of material properties
Environmental data	Temperature drop related movements; material properties; maintenance and repair strategies
Construction and maintenance records	Costs; material properties and layer thicknesses; maintenance and repair strategies
Results and experience from previous designs and analyses	Material properties; design strategies; maintenance and repair strategies

information such as surface deflection data, it forms a powerful design tool. Condition survey information is especially useful for homing in on important design inputs, such as the remaining life of the existing pavement, as will be illustrated in Chapters 3 and 4. The various kinds of distress manifestations, together with the stochastic nature of distress development, make it very difficult to translate the present condition of a pavement into design inputs. The current technology in the pavement field is imperfect. Therefore, the importance of collecting feedback information from in-service pavements is apparent. Before any models or analytical techniques can be explored, it is imperative that the results of the surveys themselves uniform and reliable. This means that condition surveys should be carried out according to well defined guidelines with detailed definitions of distress types and their degree or class.

Procedures and Forms

Most developments regarding condition survey procedures and forms took place on the network level (Ref 3 and 8). It is obvious that condition surveys at the project level should be more detailed, in terms of both quality and quantity. It is also important for results from project level surveys to tie in with network level results. The approach recommended in the design manual is

(1) to use the network level procedures and forms as a basis and to recommend extensions to these basic forms to accommodate the needs of the specific project, which for example, may include a visual assessment of drainage problems, shoulder erosion, etc;

(2) to improve the quality of the basic information required for network level analysis;

(3) to use the uniformity of the section to determine the size of base elements for the survey; and

(4) to record the location of the distress more accurately, especially when this information is used to supplement the results of special evaluation techniques, such as deflection studies. The distress

type and location can, for example, be sketched on an opaque plastic.

Because of the differences in distress manifestations characteristic to the two basic rigid pavement types, slightly different procedures, forms and data reduction techniques have been developed for continuously reinforced concrete pavement (CRCP) and jointed concrete pavements (JCP).

The condition survey procedure and field sheet of small sections (Ref 3) have been adopted for CRCP in the design manual. The network level condition survey form for JCP and JRCP was adopted from the literature (Ref 3).

Data Reduction. Information such as the location of pumping, drainage problems, etc. is used directly to supplement other evaluation techniques, and no further data reduction is required. Because of a lack of analytical techniques, condition survey information is generally not fully exploited in present overlay design procedures. It is, however, difficult to combine qualitative judgment with sophisticated numerical analysis. Gutierrez de Velasco et al (Ref 3) demonstrated how condition survey information can be used on the network level for rehabilitative scheduling. They defined a distress index in the form of a "Z"-value to reflect the important distress manifestations in a single quantifiable number. In this study the "Z"-value was used as one of the important variables to predict the remaining life of the existing pavement, as described in Chapter 3. It is thus recommended in the design manual that the basic condition survey information at the project level be reduced to the "Z"-value.

DEFLECTION DATA

Deflection data are used directly and indirectly to define various design inputs. Direct application includes material characterizations, selection of design sections, etc. When deflection data are used for void detection, the estimation of load transfer at cracks and joints, and the evaluation of the effectiveness of the repair strategy, the results are reflected indirectly in the design model. Guidelines and step-by-step procedures to collect and analyze Dynaflect deflections on rigid pavement are well summarized by Uddin et al (Ref 9). Different factors which influence deflections on rigid pavements are identified, and their effects are quantified and discussed. Guidelines for the selection of a minimum sample size of Dynaflect deflections for rigid pavement evaluation are also presented in the report by Uddin et al. The important results of this report which are related to overlay designs are

(1) The influence of temperature differential on deflections measured in the wheel paths and in the center of the slab is practically insignificant.

(2) Errors involved in deflections measured at the pavement edge are of practical significance, and temperature corrections may be required. The time of testing becomes important.

(3) No significant change in Dynaflect deflection values due to seasonal variations was measured on CRC pavements. In contrast, jointed concrete pavements showed statistically significant changes in the maximum Dynaflect deflections due to seasonal variations.

(4) The position of any non-destructive testing device with respect to the pavement edge and transverse crack or joint will greatly influence the measured deflection. In other words, when deflection data are used to make statistical inferences, the data collected at different distances from the pavement edge should not be combined.

(5) The presence of voids results in on increase in deflections. When the Dynaflect is moved toward the center of the slab, deflections decrease, and, at 5 foot from the pavement edge, there is practically no effect of void size on deflection values. An important conclusion is that, if the Dynaflect is used for material characterization, the Dynaflect should be positioned away from the pavement edge to eliminate the effect of voids.

(6) Test loads applied near pavement discontinuities result in higher deflections than the corresponding deflections measured away from the discontinuity.

(7) Placement error should be kept as small as possible and should ever exceed 5 inches.

(8) Replication error is generally below 10 percent for the Dynaflect.

(9) If any rigid layer exists at some depth, deflection measurements and subsequently Young's modulus of the subgrade will be affected significantly and should be accounted for.

(10) The variation of thickness of the concrete PCC slab is a source of variation in deflection data. A change in slab thickness of $\pm$ 0.25 inch typically causes a variation of approximately 2.5 percent in the Dynaflect sensor 1 deflection.

These results were considered in developing the guidelines for deflection testing presented in the design manual. Well defined procedures and guidelines on aspects such as positioning, temperature corrections, time of testing, etc. for specific applications of deflection data are presented in the design manual.

Reduction techniques for deflection data are a function of the specific application of the data and are discussed in sufficient detail in the appropriate sections of the design manual, which is Appendix A.

OTHER SOURCES OF DESIGN INFORMATION

The other sources of design information mentioned in Table 2.1 are self explanatory and discussed in sufficient depth in the design manual, which is presented in Appendix A.

CHAPTER 3. CHARACTERIZATION OF THE EXISTING PAVEMENT

Mechanistic design procedures require a suitable theory and model to analyze and predict the behavior of a pavement structure. Plate, elastic layer, and finite element theories have been used for this purpose. Typically, these theories are used to compute the tensile stresses or strains in the upper, bound, pavement layers, which are then entered into a fatigue equation to predict the life of the pavement. Since elastic layer theory computer programs are readily available and relatively easy and cheap to use, layer theory has been adopted in the Texas Rigid Pavement Overlay Design System (Ref 4). The shortcomings of the theory, such as the inability to predict pavement responses under an edge loading condition, are corrected by using appropriate critical stress factors developed from finite element theory. The output of the mechanistic model is, however, a direct function of the design inputs, which, in the case of elastic layer theory, are Young's moduli and Poisson's ratio for the pavement layer. The accuracy of the model predictions will, thus, be a function of how well the pavement structure is represented by the material properties used in the model.

The design of pavement rehabilitation has a major advantage over the design of new pavements, because many of the design inputs which have to be assumed or estimated for a proposed pavement can be measured with a certain degree of accuracy on existing pavements. These include in situ moduli bearing strengths and traffic. Rehabilitation can, in fact, be regarded as a modification of the behavior of an existing pavement, with known strengths and weaknesses. The more fully this behavior is evaluated the more accurate and, thus, the more economical should be the rehabilitation. However, much of the information that can be obtained by evaluating the conditions of the pavement and its past behavior is generally not fully exploited. Although these deficiencies may be recognized, it is often difficult to combine qualitative judgment with sophisticated numerical analysis.

In Chapter 2 the important sources of design information are identified. This chapter deals with the development of techniques to combine these sources of information to arrive at representative design inputs. More specifically, the determination of the elastic properties and remaining life (translated into remaining fatigue life) of the existing pavement are discussed. The relationship between the elastic properties and the remaining fatigue life provided by the mechanistic model is explored in an attempt to "calibrate" the mechanistic model by comparing the model predictions of the past behavior to the actual distress conditions of the pavement.

ELASTIC PROPERTIES OF THE EXISTING PAVEMENT

Four sources of information are available to estimate the elastic properties of the existing pavement layers. These are

(1) laboratory test results of samples taken from the design section,
(2) typical properties of similar materials in the region (obtained from previous laboratory tests and analyses),
(3) deflection data, and
(4) present condition of the pavement.

The first two sources are self explanatory, and tables summarizing the recommended laboratory tests and typical material properties extracted from literature reviews by the author are presented in the appropriate sections of the design manual, Appendix A. This paragraph describes how deflection data can be combined with laboratory test results and results from previous tests and analyses to arrive at a set of elastic moduli of the existing pavement layers. The material presented is mainly a summary of previous research findings (Refs 2 and 9). The shortcomings of the procedure are discussed. The development of techniques to alleviate some of these shortcomings and how the overall material characterization procedure is accommodated in the design manual are described later in this chapter.

Dynaflect Deflections

Several deflection testing devices (Ref 10) are available on the market. The Dynaflect testing device is generally used in the State of Texas and most current material characterization procedures are associated with this device. Chapter 2 described the development of recommended procedures and basic data reduction techniques for Dynaflect measurements on rigid pavements. The next logical step is to use the data to characterize the existing structure. Elastic layer theory is applied to analyze the Dynaflect deflections for material characterizations. Taute et al (Ref 2) did useful work in this regard and further work by Torres-Verdin et al and Uddin et al (Refs 5 and 10) concentrated on detailed aspects of the procedure such as sources of error and, the effect of environment and position of measurement on surface deflections. Procedures for the necessary corrections were developed in these studies. The results of several studies were finally incorporated in a user manual for Dynaflect testing on rigid pavements (Ref 9). The procedure adopted in the overlay design manual is based mainly on the latter report. The step-by-step procedure is presented in detail in Appendix A, and no further discussion is devoted to the mechanics of the procedure. The applicability and shortcomings of the procedure and how it relates to overlay design are discussed in more detail in the rest of this paragraph.

The shortcomings of the material characterization procedure can be classified into two groups. These are

(a) the problem and errors associated with the actual deflection testing, and
(b) problems associated with the use of the deflection data to determine the elastic material properties.

Procedures to correct or avoid some of the first group of shortcomings are discussed in sufficient depth in Chapter 2 and Ref 9. The problems and potential errors associated with the use of surface deflection data to determine elastic material properties are discussed below.

Elastic Material Properties from Dynaflect Deflections

The rigid pavement structure is modeled as a multi-layered linearly elastic system with homogeneous and isotropic material within each layer. An iterative procedure is then used to arrive at a set of elastic moduli that will fit the measured surface deflections to those predicted by elastic layer theory. The following limitations have been identified:

(1) The iterative procedure does not provide a unique solution.

(2) The presence of a rigid layer at shallow depth below the subgrade can potentially lead to errors in the prediction of subgrade moduli.

(3) Variations (both random and stratified) remain in the deflection results, which complicates the selection of design deflection values.

In light of the limitations mentioned above, it is clear that the design of pavement rehabilitation sets high demands the design engineer. Exploiting the information that can be obtained by analyzing the past behavior and the current condition of the pavement can thus potentially be very useful in supporting the judgment of the design engineer. This is particularly true in the final selection of the modulus ratios in the iterative procedure used for deflection basin fitting. The development of a "calibration" technique for this purpose is discussed in the last part of this chapter. Taute et al (Ref 2) did useful work which enables the design engineer to consider the presence of a rigid layer at shallow depth below the subgrade. The appropriate graphs and techniques recommended in Ref 2 are incorporated in the design manual. Variation in deflection data within a design section is another important aspect to consider. Where possible, stratified (or assignable) variation is accounted for by separating design sections with assignable differences. This is not always practical; for example, when a small stretch of weak subgrade is encountered; it is normally included in a larger section, for practical reasons, and its variation is added to the random variation of the larger section. Random variation must be accounted for by designing for deflections on the basis of a certain statistical confidence limit. The subgrade modulus generally has a large amount of variation associated with it, and previous studies (Ref 1 and 2) have recommended that the pavement be designed for a certain confidence limit only with regard to this layer. Due to the correlation between the subgrade modulus and the Dynaflect sensor 5 (W_5) deflection, this may be interpreted as designing for a confidence limit with regard to the W_5 deflection. The 90th percentile W_5 deflection is used in the design manual, following the practice adopted in the previous studies (Refs 1 and 2). The modal deflection basin slope (W_1 - W_5) slope is recommended for use in the material characterization of the upper bound layers. The rationale behind this recommendation is that less variation occurs in these layers. Furthermore, it was clear from previous studies (Ref 2) that the modal (W_1 - W_5) deflection slope approximately corresponds to the 85th to 90th percentile deflection slope. Extreme deviations from this value are accounted for through repair measures and the selection of appropriate stress factors. Sensitivity analyses also proved that the fatigue analysis is sensitive to variations in layer thicknesses and concrete flexural strength values, and some degree of conservation can, if necessary, be applied to these variables. A large proportion of the variation associated with the basin slope measurement may result from changes in the subgrade modulus (Ref 2). Therefore, it is recommended that the subgrade modulus to be used in the calculation of the upper layer moduli (from the basin slopes) be determined from the same W_5 statistic used with the basin slope, i.e., the modal W_5 deflections.

REMAINING LIFE OF THE EXISTING PAVEMENT

The optimal rehabilitation strategy under certain conditions, such as high traffic growth rates, may be to place an overlay before the existing pavement has reached the end of its structural life. The remaining structural life of the existing pavement must then be quantified. The design procedure models remaining life as remaining fatigue life in 18-k ESAL's.

The concept of remaining life is easy to understand, as is the modeling of remaining life as a fatigue concept. Remaining life (RL) is simply calculated using

$$RL = (1 - \frac{n_{18}}{N_{18}}) * 100 \qquad (3.1)$$

where

RL = remaining fatigue life expressed as a percentage,
n_{18} = accumulated past traffic in 18-k ESAL's, and
N_{18} = initial structural design life in 18-k ESAL's.

Because of uncertainties associated with the determination of n_{18} and N_{18}, the quantification of remaining life is rather complex. Before the quantification of remaining life can be discussed any further, the concept of structural failure must be explored.

Structural Failure

Before pavement life can be determined, or predicted, the pavement condition that constitutes "failure" must be defined. Recognizing this, the Present Serviceability Index (PSI) concept was introduced during the AASHO Road Test. A terminal condition, at which the pavement is said to have failed, based on the level of service to the user, was defined. This type of pavement failure can be termed "functional failure." The point of functional failure does not necessarily correspond to the point of structural failure, as illustrated in Fig 3.1. This is probably due to continuous maintenance carried out on existing roads. To accommodate the failure concept in mechanistic rehabilitation design models, the point of structural failure needs to be defined, as it will be modeled as the point in the life of the pavement at which it reached the end of its fatigue life. Recognizing this, the concept of pavement function was extended in previous studies (Ref 2) to

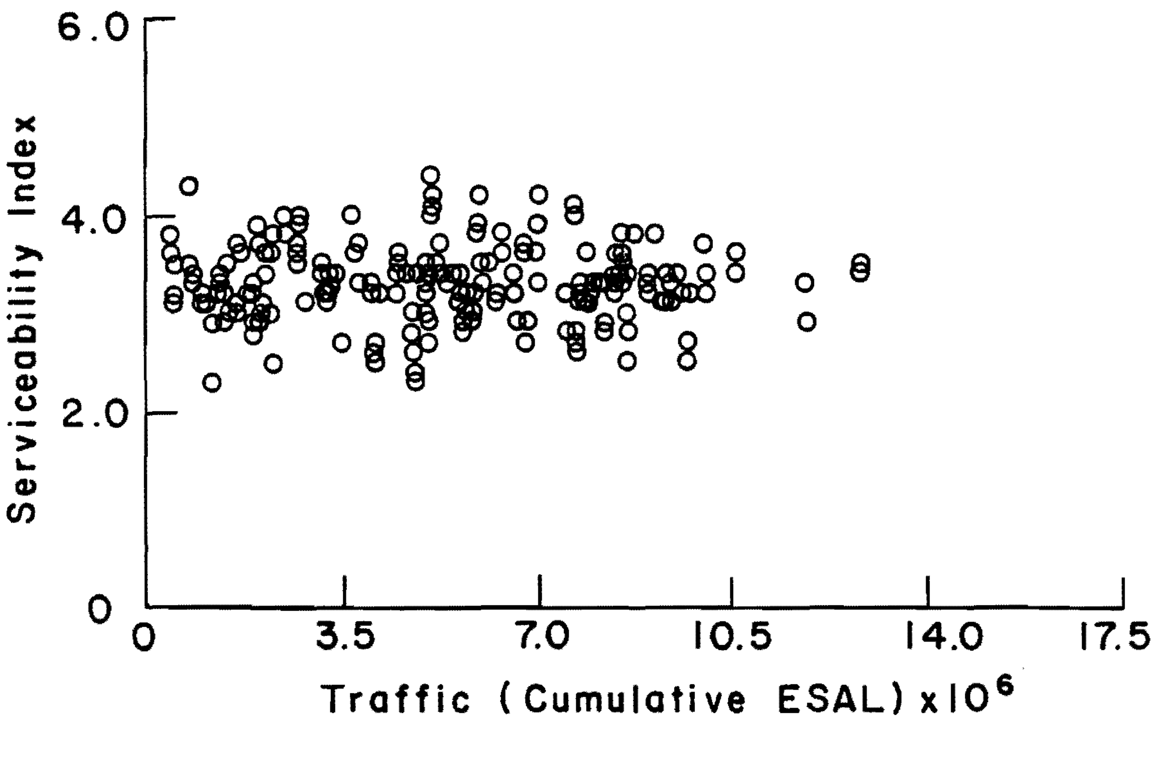


Fig 3.1 **Serviceability index versus traffic applications (both directions) for Texas CRCP sections surveyed in 1974 and 1978.**(Ref 3)

the ability of the pavement to serve the user as economically as possible. Taute et al (Ref 2) used this concept to develop an improved fatigue equation for use in Texas after reviewing AASHO Road Test data and Texas condition survey results. A fatigue equation generally represents some terminal condition, depending on the data used to develop the equation. Before any fatigue equation is used for design purposes, this fact must be recognized and considered. Taute et al defined the point of structural failure as that point in the life of the pavement after which any further traffic loading would result in a rapid increase in distress. The assumption was that this point in the life of the pavement represents the end of the structural fatigue life. He also demonstrated that this point corresponds approximately to the "economic failure" of the pavement by weighing maintenance and rehabilitation costs. The end of the structural life of the existing pavement as predicted by the fatigue equations adopted in the overlay design procedure is thus linked to the rate of distress development. This link was further explored in this study to develop relationships between present condition of the pavement and remaining life.

Relationship Between Pavement Condition and Remaining Life

The RPOD2 design procedure uses a fatigue equation developed from the AASHO Road data to make predictions of the pavement life (Ref 1). The terminal condition of the pavement was considered to be the initialization of Class 3 and 4 cracking. Class 3 cracking is defined as significantly spalled cracks which are approximately 1/4 inch wide and Class 4 cracking is any crack which has been sealed. Pavements that exhibited Class 1 or 2 cracking, or no cracking, were assumed to have remaining life. The remaining life was quantified using elastic layer theory and assuming that the elastic material properties obtained from surface deflection data are accurate. Recognizing the limitation of this assumption, Taute et al (Ref 2) introduced the new structural failure concept and modified the fatigue equation used in RPOD 2. Based on Texas condition survey data, Taute et al (Ref 2) developed the following approximate rules of thumb to identify the point of structural failure for CRC-pavements.

(1) The critical rate of defect development is in the order of three defects per mile per year at the point of structural failure.

(2) The point of structural failure can be defined approximately as the condition where the number of defects per mile equals the pavement age in years.

They also used the present distress condition of the pavement to obtain representative elastic moduli when severe discrepancies existed between mechanistic predictions of past structural behavior and the actual distress condition of the existing pavement. For this purpose they broadly classified the distress condition as "minor" or "severe." The above concept is refined in this study, as described in the rest of this section.

Distress Prediction Equation

Before the relationship between distress (and rate of defect development can be further explored, it is necessary to be able to predict distress. Due to the complexity of considering all the factors involved in distress development, such as pavement structure, traffic, and environmental conditions, plus construction and maintenance variables, the existing prediction equations rely more on empirical results and engineering judgment than theoretical concepts. Machado et al (Ref 11) and Potter (Ref 12) developed failure prediction equations using the 1976 CRCP condition survey data. Noble and McCullough used the 1978 data to update these equations (Ref 13). Gutierrez de Valasco et al (Ref 3) checked these equations using 1980 condition survey data and concluded that they tended to over predict. They developed a new set of equations which were finally incorporated into the network level analysis (Ref 3). Separate equations were developed for three typical distress manifestations on CRC pavements, namely, failures (i.e., punchouts and patches), minor spalling, and severe spalling. The equations assume that information about the distress was obtained at some time in the life of the pavement. This information, together with the pavement age at the time of the condition survey and the age at the time chosen for distress prediction, enters the prediction equation. It was, however, considered necessary to include more variables in a project level prediction equation. Strauss et al (Ref 14) developed distress prediction equations for CRCP using theoretical formulations and field information. However, these equations included many variables and, in light of presently unavailable information, they were not considered practical for implementation at this stage. It was decided to develop a new equation that would meet the following requirements:

(1) The variables included in the equation should generally, be, available at project level.
(2) The equations should, preferably include important variables such as traffic and environment to improve the prediction accuracy.
(3) Rate of defect development should be predicted since it is, at this point, the best link between distress and fatigue life.

After careful study of the available condition survey data and the results of previous studies (Refs 3, 8, 13 and 15), the following variables were considered in the preliminary analysis:

(1) the distress Index (Z value) of the existing pavement, as implemented in the network level analysis (continuous variable).
(2) the existing pavement age (A) in years as a categorical variable at three levels: $A \leq 10$; $10 \leq A \leq 15$; $A \geq 15$.
(3) the equivalent past traffic (T) in 18-k ESAL's as a categorical variable at three levels: $T \leq 4 * 10^6$; $4 * 10^6 < T \leq 8 * 10^6$; $T \geq 8 * 10^6$.
(4) the district in which the pavement is located, as a qualitative variable. The Districts which have been included in the analysis are shown in Fig 3.2.

An analysis of variance (ANOVA) was done using a data set of 242 points. The rate of defect development per mile over a 4-year period (1978 to 1982), pavement age (A) and past traffic (T) in 1982, and District (D) were

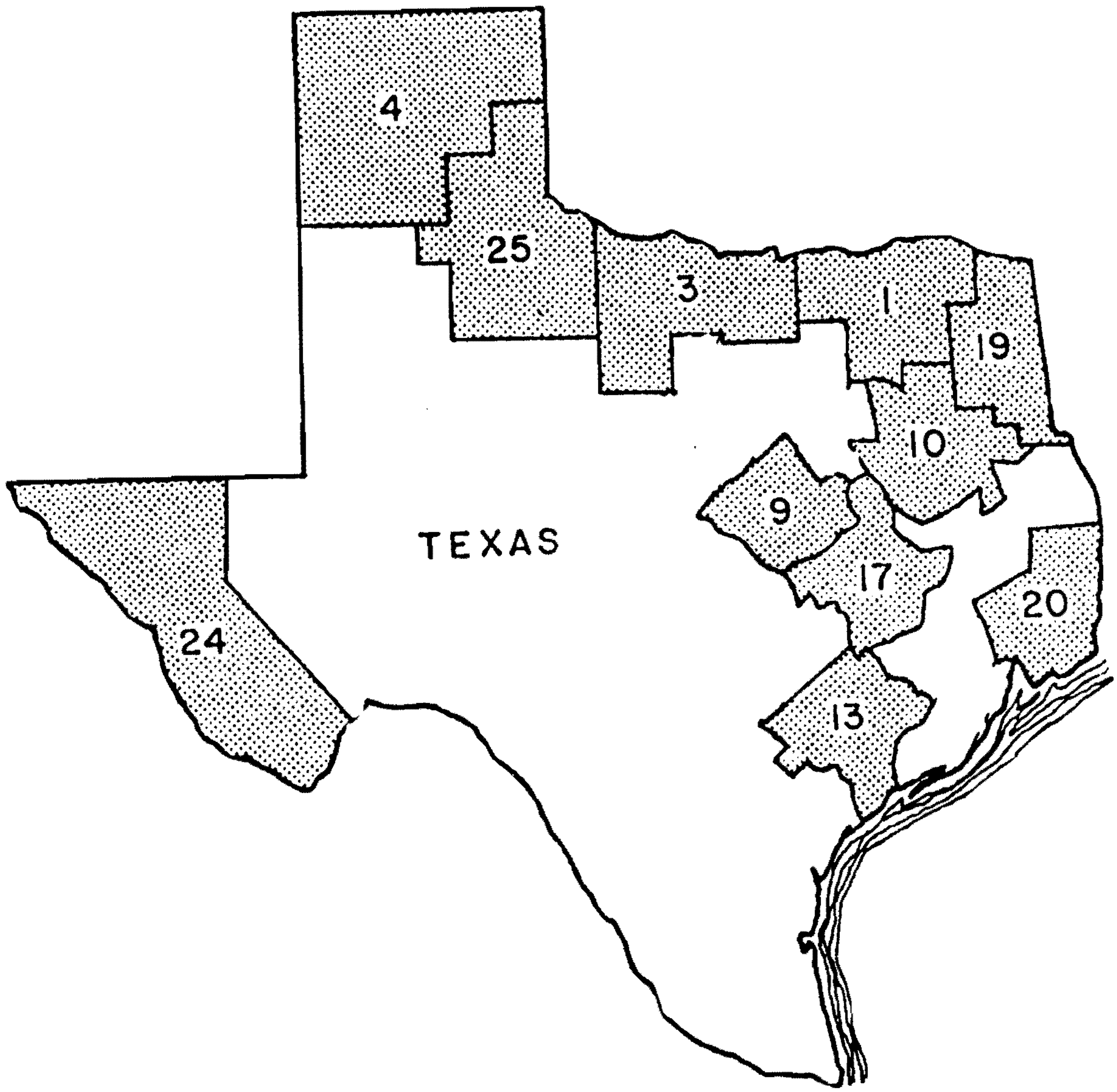


Fig 3.2. Texas rural districts surveyed to collect CRCP information (Ref 3).

considered in the ANOVA. From the ANOVA table, the following conclusions were evident:

(1) D and A were significant at 5 percent α level,
(2) T was not significant at 5 percent α level, and
(3) no two-way interaction was significant.

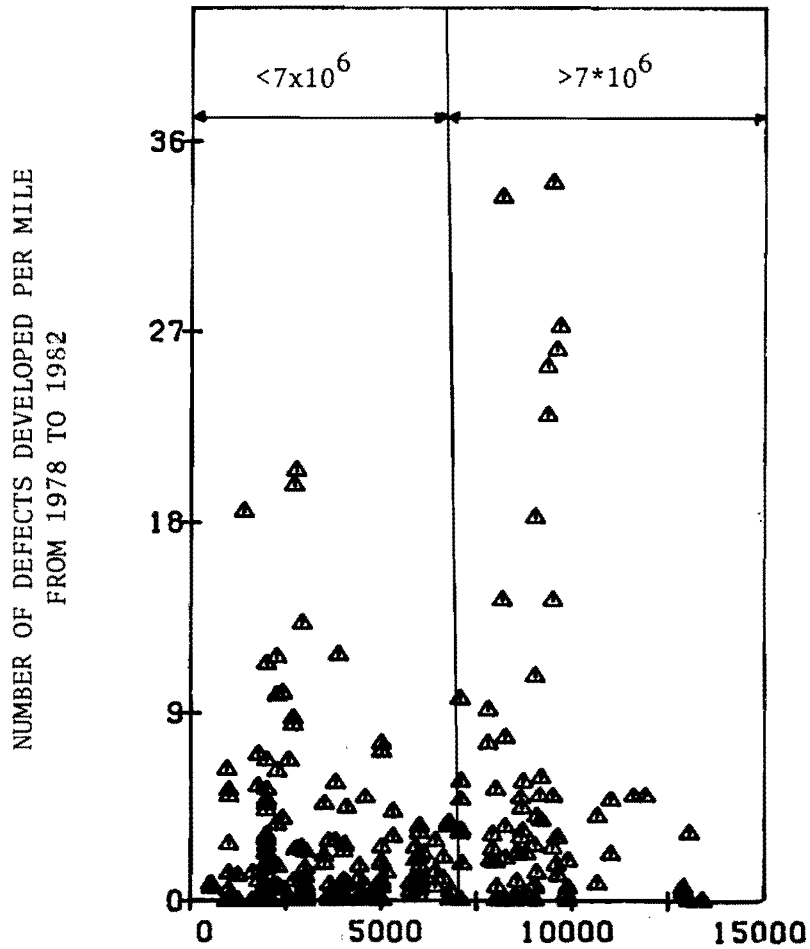
With the Distress Index (Z) as a covariate, an analysis of covariance was done on the data set. The 1982 condition survey data was used to determine the Z-value. The following conclusions were evident from the analysis of covariance table:

(1) D was significant at 5 percent α level,
(2) T and A were not significant at 5 percent α level,
(3) no two-way interactions were significant, and
(4) Z was significant at 5 percent α level.

Figures 3.3, 3.4, 3.5 and 3.6 support the results of these analyses. Based on the results of the above analyses, it was decided to consider age and traffic, only, at two levels. The levels considered in the further analysis are also shown in Fig 3.3 and 3.4.

In order to develop a meaningful relationship, the rate of defect development (over a four-year period) was used as the dependent variable; the Distress Index (Z) as the quantitative variable; and traffic (T) and age (A) as dichotomous (dummy) variables at two levels and District (D) as a categorical variable in a multiple regression analysis. The regression technique used is very well documented by Uddin (Ref 10). The regression equation that resulted from the regression analysis is shown in Fig 3.7. The R^2 statistic for the equation is 0.67.

<u>Discussion of Distress Prediction Equation</u>. The prediction accuracy of the equation is illustrated in Fig 3.8. The rate of defect development in the figure is the number of failures developed over the 4-year period from 1978 to 1982. It is clear from Fig 3.8 that the equation is under predicting when the rate of defect developments becomes high. This was, however, not



CUMULATIVE PAST TRAFFIC (18k ESALS x 1000)

Fig 3.3. Relationship between cumulative past traffic and number of defects developed from 1978 to 1982.

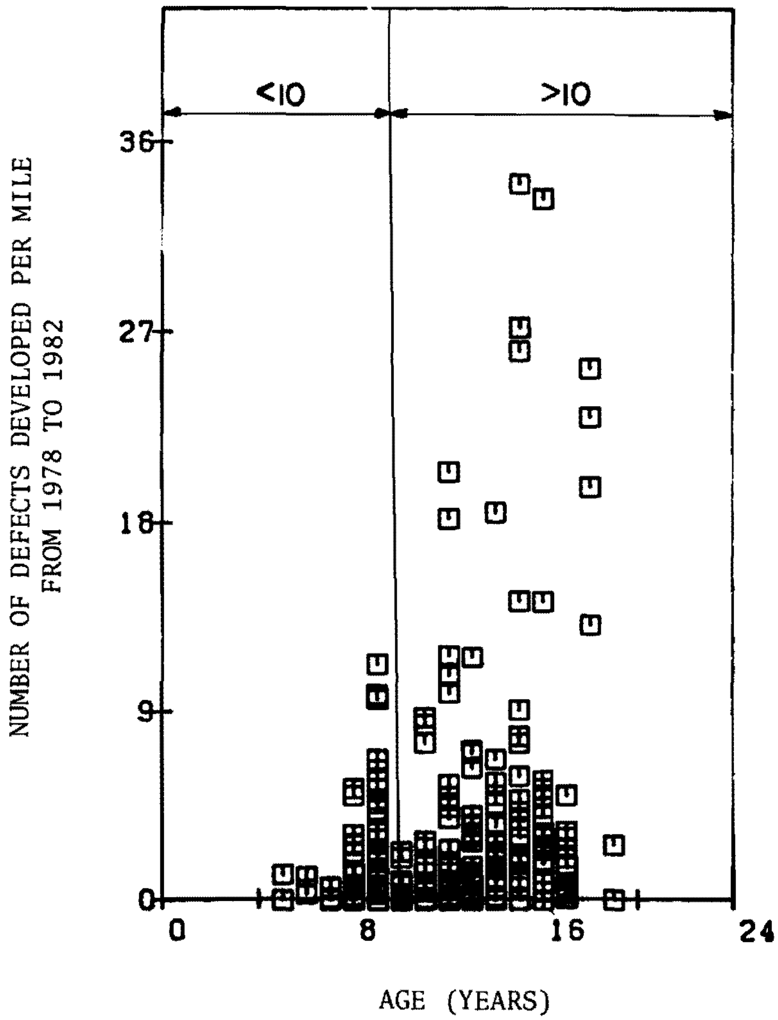


Fig 3.4. Relationship between pavement age and number of defects developed from 1978 to 1982

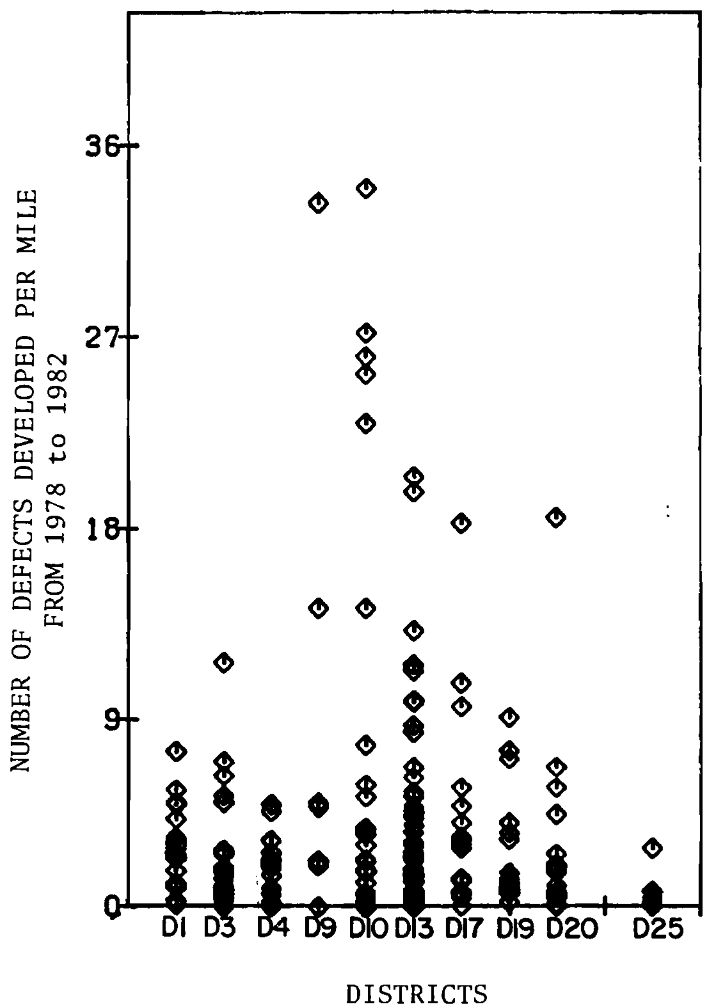


Fig 3.5. Relationship between district and number defects developed from 1978 to 1982

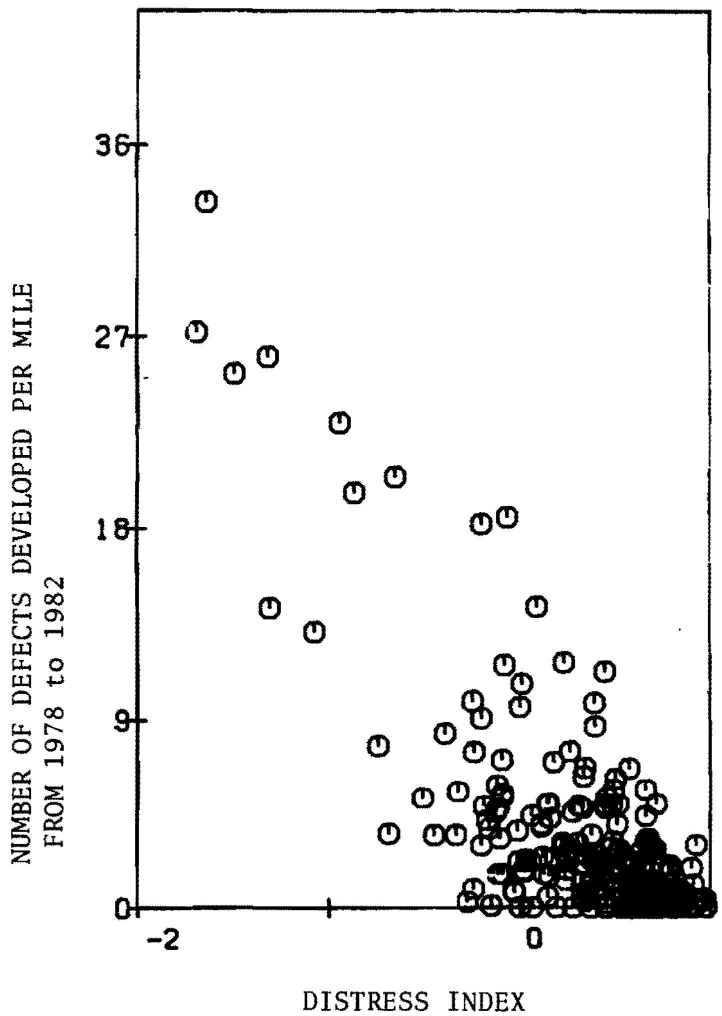


Fig 3.6. Relation ship between distress index and number of defects developed from 1978 to 1982

NUMBER OF DEFECTS PER MILE DEVELOPED FROM 1978 TO 1982 = Σ (VARIABLE X COEFFICIENT)

VARIABLE	COEFFICIENT	RANKING	R^2
Z	- 9.8838982	1	0.67
D 1	- 1.0744446	10	
D3	1.0768425	6	
D 4	0.19846003	13	
D 9	1.5087324	7	
D 10	- 2.8620093	2	
D 13	- 1.2498002	3	
D 17	1.1817181	9	
D 19	- 0.39732579	12	
D 20	1.1497876	5	
D 24	- 1.5642249	8	
D 25	0	-	
A	0.85278825	4	
n	0.38803879	11	
INTERCEPT	6.8437148		

Z = DISTRESS INDEX

Di = DISTRICT: Di = 1 if Di = Di ; Di = 0 if Di ≠ Di

A = AGE: A = 1 if A > 10 YEARS

A = 0 if A ≤ 10 YEARS

n = CUMULATIVE PAST TRAFFIC (in one direction):

n = 1 if n > 7 million ESALS

n = 0 if n ≤ 7 million ESALS

Fig 3.7. Regression equation to predict the number of defects developed per mile from 1978 to 1982.

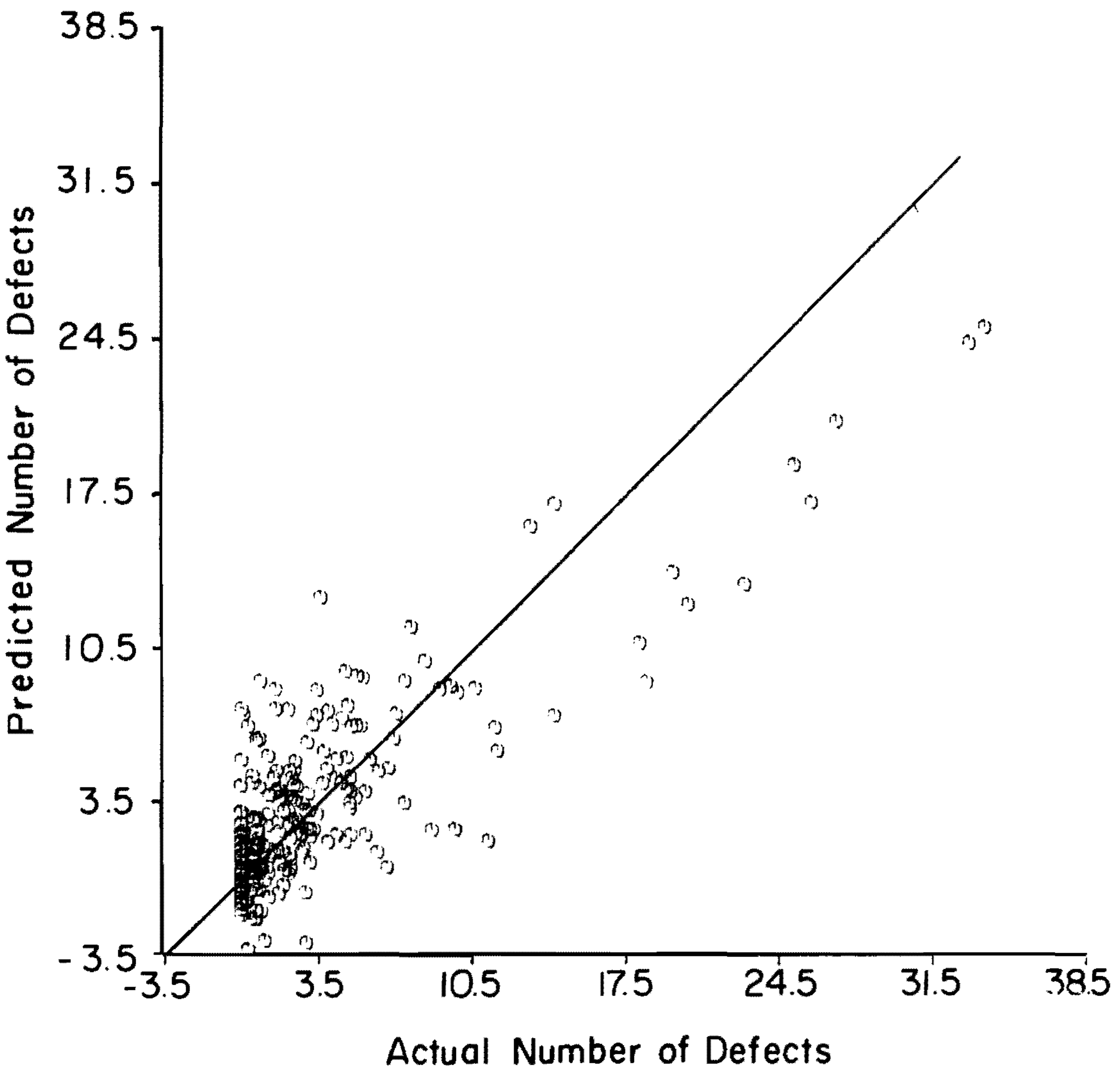


Fig 3.8. Illustration of the prediction accuracy of regression equation predicting number of defects developed per mile from 1978 to 1982

considered to be a serious limitation since zero remaining life would be predicted for these sections. The ranking of the individual variables is shown in Fig 3.7. The Distress Index is the most important variable, followed by some of the Districts. The R^2 statistic was considered reasonable in light of the available data and the variables considered. It is, however, clear that more research is required in the area of distress prediction, particularly for project level application.

Relationship Between Distress Rate and Remaining Fatigue Life

The next step was to relate rate of defect development to remaining fatigue life. First, the rate of defect development over the 4-year period (1978 to 1982) was converted to a yearly rate. The failure prediction model developed by Gutierrez de Velasco et al (Ref 3) was used to develop the required relationship for the relevant age categories. Previous study results (Ref 2, 3 and 41) and further study of individual projects for which more information about the structural performance was available, led to the set of decision criteria presented in Table 3.1. Using the regression equation (Fig 3.7) and the decision criteria (Table 3.1), a nomograph (Fig 3.9), was constructed for remaining life estimates as a function of Distress Index and Districts, for the four combinations of age and past traffic, and was incorporated in the design manual.

CALIBRATION OF DESIGN MODEL

Two independent estimations of the remaining life of the existing pavement can now be made. The first method estimates the remaining life using a regression equation which includes the following variables:

(1) Distress Index (Z value),
(2) pavement age group,
(3) accumulated past equivalent traffic, and
(4) district in which the pavement is located.

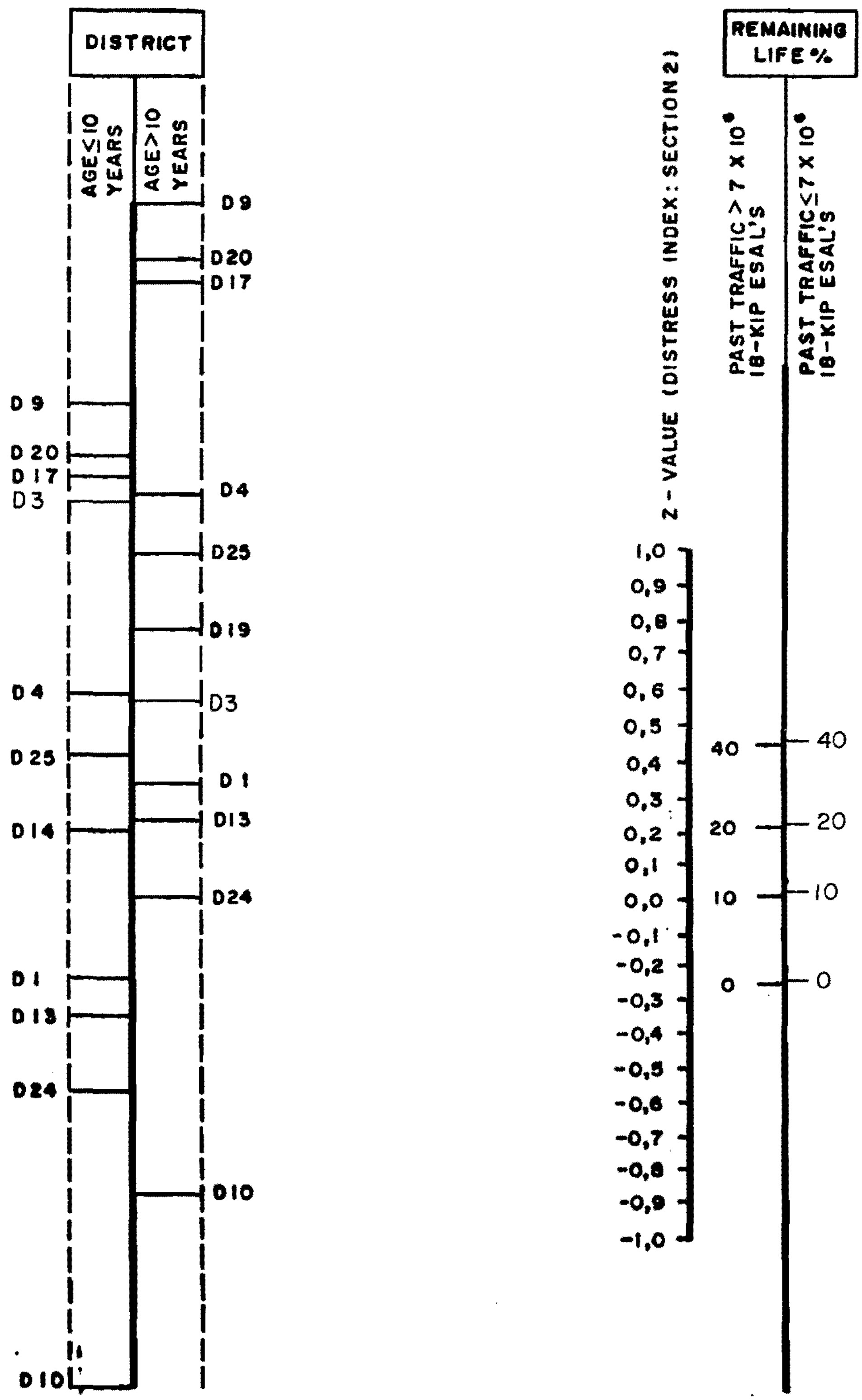


Fig 3.9. Nomograph to determine life as a function of district age, distress index, and accumulated past equivalent traffic.

TABLE 3.1. DECISION CRITERIA USED TO DETERMINE REMAINING LIFE FROM RATE OF DEFECT DEVELOPMENT

Yearly Rate Per Mile of Defect Development	Remaining Life (Percent)
> 3.0	0
2.5	10
1.8	20
< 1.0	40

The second method uses the preliminary set of elastic moduli determined from deflection data to calculate the critical response of the existing pavement. The appropriate fatigue equation is then used to calculate the original design life of the existing pavement in 18-kip ESAL's. The accumulated past traffic on the existing pavement is then estimated and the remaining life determined using Eq 3.1.

Major discrepancies between the two estimates of the remaining life of the existing pavement should be investigated and adjustments made with the following points in mind:

(1) The determination of elastic moduli (especially the modulus ratios of the upper layers) from surface deflection data does not provide a unique solution.
(2) The presence of a rigid layer at shallow depths should be accounted for.
(3) The critical stress factor used to account for discontinuities and voids should be accounted for (refer to Chapter 4).
(4) The first method for remaining life predictions (regression equation) is approximate and can point out only major discrepancies.

The method as outlined in Fig 3.10 should avoid major discrepancies between model predictions and structural performance. The method is far from perfect and should be improved when more data become available. Condition survey data on JC pavements in Texas are initially very limited, and the nomograph in Fig 3.9 was developed from early CRCP condition survey data.

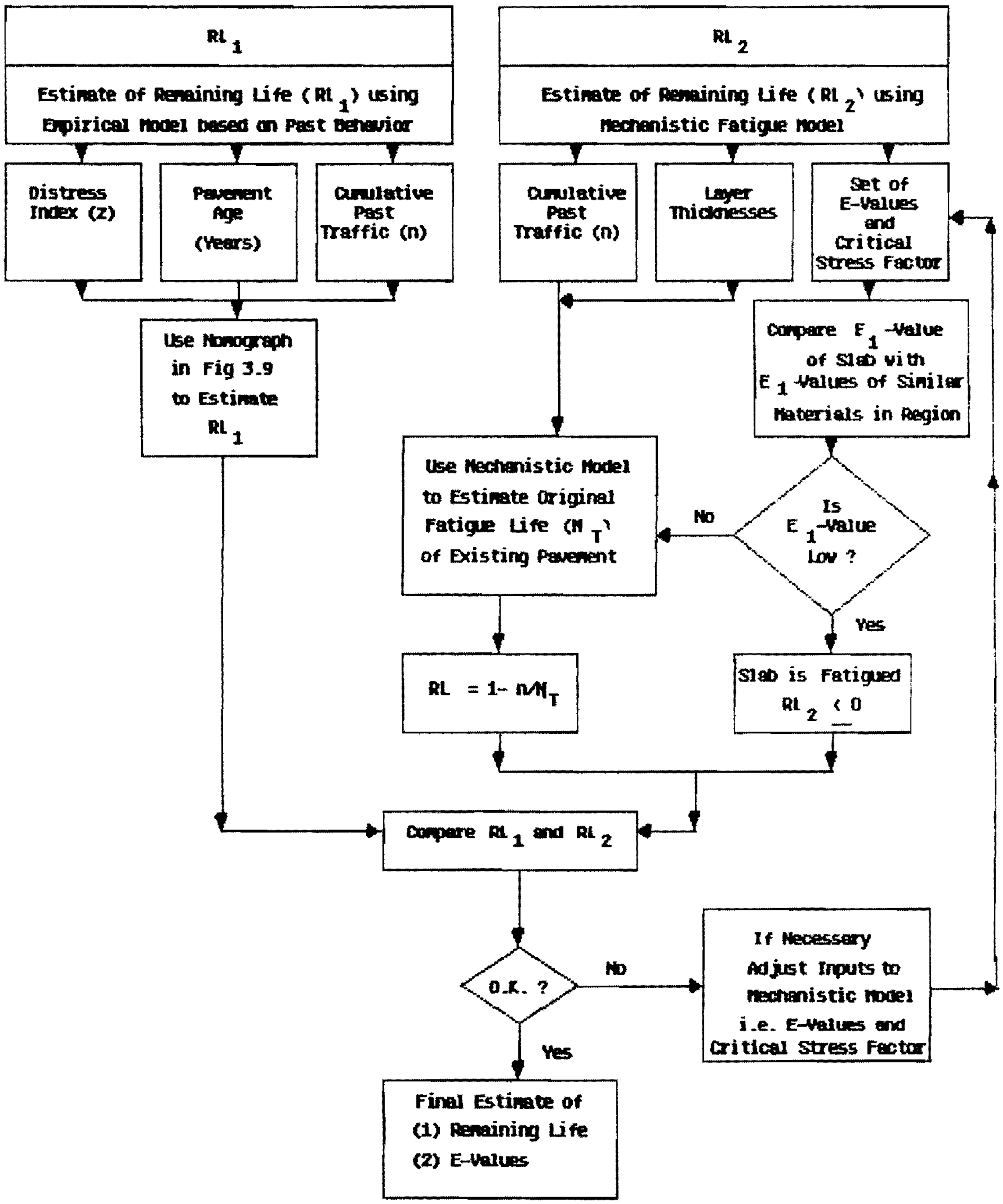


Fig 3.10. Recommended procedure for the estimating of remaining life and the calibration of the mechanistic model.

CHAPTER 4. REHABILITATION DESIGN STRATEGIES

Variables, such as different overlay types and techniques, condition of the existing pavement, timing of overlay(s), maintenance and repair techniques, etc., can potentially generate a large number of rehabilitation design strategies. For the purpose of discussion in this chapter, rehabilitation strategies are broadly classified into two groups, namely (a) basic overlay strategies, which includes overlay type and timing of overlay placement, and (b) repair strategies, which include repair and preparation of the existing pavement prior to overlay placement.

BASIC OVERLAY STRATEGIES

There are several types of design constraints which differentiate between overlay design strategies which are feasible and those which are not feasible. Constraints that are generally considered in overlay design include

(1) available funds,
(2) policy constraints,
(3) minimum allowable time to first overlay,
(4) minimum allowable time between overlays,
(5) length of analysis period or minimum life of strategy, and
(6) maximum and minimum asphaltic concrete (AC) or Portland cement concrete (PCC) thicknesses.

Computer program RPRDS-1 (Ref 4) contains a routine called STRTGY which generates possible overlay design strategies. All the constraints mentioned above can be considered either directly or indirectly by RPRDS-1.

The program provides the user with cost information for an extensive list of alternative strategies which are characterized in terms of timing of overlay placement and overlay material type. The first two constraints, available funds and policy constraints, can thus be considered indirectly since the long-term consequences of fund allocation and policy decisions can be evaluated.

The third constraint, minimum allowable time to first overlay placement, can be considered indirectly since the user can specify certain levels of remaining life at which the first overlay may be placed.

The fourth constraint is considered directly in the program, and the user can specify the minimum time between successive overlays.

The fifth constraint is handled in a fairly unique way by the program. The user is allowed to specify some maximum period of heavy maintenance, which, in effect, permits consideration of those strategies that do not quite last the analysis period without some period of heavy maintenance.

RPRDS-1 (Ref 4) requires that the designer select the specific thicknesses of AC and PCC overlay to be considered. The maximum allowable total overlay thickness must also be specified by the user. The sixth constraint is thus considered directly in the program.

<u>Development of Strategy Tables</u>

The capabilities of the program were exploited to develop a series of tables to present the user of the design manual with the likely relative cost of different basic design strategies under different conditions. Both JCP and CRCP's were considered in the analysis. A 3 x 3 x 4 factorial was designed for each pavement type. Traffic and subgrade modulus were considered at three levels, and four levels of present remaining life of the existing pavement were analyzed. The other variables were fixed at values typical for the State of Texas. The input data used in this analysis are presented in Appendix B. The levels of the variables, as well as the values of the variables fixed in the analysis, were defined after careful study of typical conditions in the State. Cost information was obtained by reviewing several bid packages from recent overlay contracts. In the specification of design constraints, practical aspects, such as minimum and maximum practical overlay thickness and the exclusion of bonded PCC-overlays at low remaining life values, were built into the analysis.

The following approximate information can be obtained from the set of tables incorporated in the design manual:

(1) a set of feasible basic overlay strategies for different conditions of subgrade, traffic, and current distress (i.e., remaining life) of the existing pavement.

(2) the relative total cost of each strategy, which allows the selection of candidate optimal strategies. The approximate consequences of not selecting the optimal strategies, for example, postponing an overlay because of short-term budget constraints, are also evident from the cost ratios.

(3) below each cost ratio the approximate contribution of user delay cost (expressed as a percentage of total cost). This allows the user to estimate delay cost from construction cost calculations.

(4) general guidelines which can be developed. For example, at low traffic levels, it is generally more cost effective to delay overlay placement to the point where the existing pavement approaches zero percent remaining life. The opposite is generally true at high traffic levels. Concrete shoulders are generally economically feasible only at high traffic levels, particularly when poor subgrade conditions are encountered.

Applicability of the Strategy Tables

The tables can be used for the screening of candidate optimal strategies under a wide range of subbase and subgrade conditions. The results of the analysis are applicable to only a small range of existing PCC-slab thicknesses, these being 7 to 9 inch for CRCP and 9 to 11 inch for JCP. Other cost sensitive variables that were fixed in the analysis are the delay model variables and production rate. These variables became very important at high traffic levels. The general guidelines will, however, be the same. Cost differences of less than 10 percent are probably not significant in light of the many variables involved. The user of these tables should, thus, not look at only a single optimal strategy.

REPAIR STRATEGY

Elastic layer theory assumes that the pavement layers are elastic, homogeneous, and isotropic. Unfortunately, real pavements, particularly rigid pavements, are not that simple. They have joints, cracks, edges, corners, non-uniform support, distress manifestations and other similar types of discontinuities, which have a large effect on pavement response. Finite element adjustment factors were developed to account for the effects of some of these discontinuities on the response of both the existing slab and the overlay. Tables of recommended adjustment factors have been developed by Seeds et al and are presented in Ref 4. These tables provide a range of values within which the factor must be selected by the user. Guidelines on how to select these factors as a function of existing pavement conditions are not well defined in the literature. The effects of "abnormal" conditions such as severe loss of support on pavement response, have been the subject of a number of special studies (Refs 2, 4, 5 and 6). Repair techniques to correct such abnormalities and the effect of the repair strategy on pavement response have been investigated. It is clear that the repair strategy will have a significant impact on the critical response of both the existing pavement and the overlay. It was, therefore, considered appropriate, in this chapter, as well as in the design manual, to consider the selection of adjustment factors as part of the discussion of repair strategy.

Stresses at Cracks under an Interior Loading Condition

Taute et al (Ref 2) used finite element theory to analyze the effects of changing crack spacing on the tensile stresses of a PCC-slab. A typical set of results is presented in Fig 4.1. The results indicated that when the surface deflection at the crack exceeds approximately 1.5 times the interior deflection of an uncracked pavement, the tensile stresses acting parallel to the crack may begin to exceed the tensile stresses in the uncracked pavement (i.e., interior stresses). The condition is aggravated by a closer crack spacing, as indicated in Fig 4.1. A deflection ratio (interior

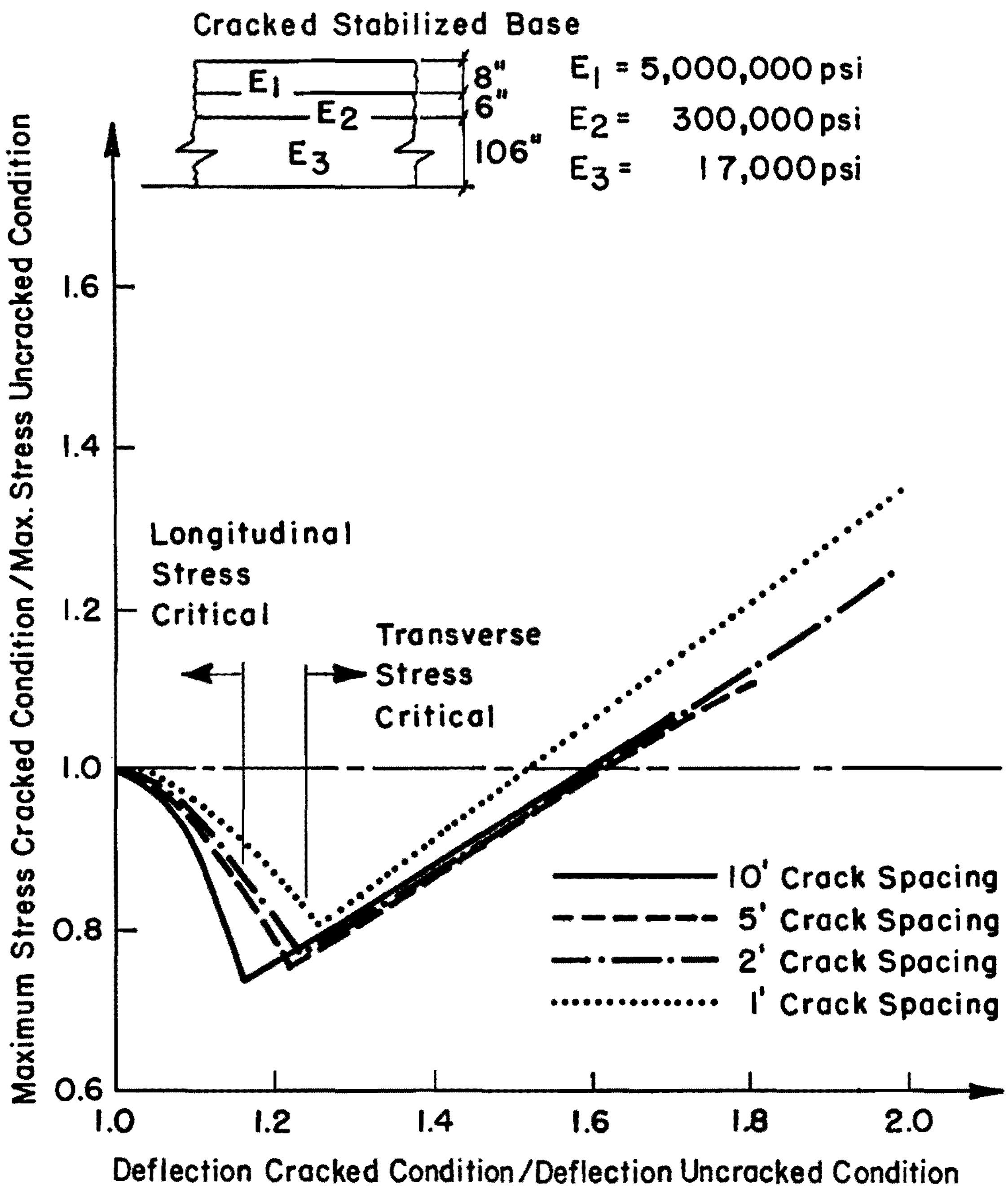


Fig 4.1. The effect of load transfer and crack spacing on the critical tensile stresses in rigid pavement for an interior loading condition (Subgrade modulus = 17,000 psi) Ref 2.

deflection/deflection at crack) of 1.5 is possible only where loss of load transfer at the crack is present. Uddin et al used the Slab 49 program (Ref 9) to develop a diagnostic chart for load transfer evaluations. A crack in a rigid pavement was simulated by reducing the slab bending stiffness. This chart was incorporated in the design manual. The results of this analysis indicated that a deflection ratio of approximately 1.6 corresponds to serious loss of load transfer and warrants special attention. The results of these studies led to the recommendation in the design manual that the cracks which exhibit significant loss of load transfer be repaired prior to overlay placement. It is clear that higher stresses than those predicted by the interior loading condition (midspan loading) may result when the deflection ratio approaches the value of 1.5.

Stresses Under an Edge Loading Condition

Taute et al (Ref 2) analyzed the stress-deflection relationship of the edge loading condition as a function of transverse crack spacing. A typical set of results is shown in Fig 4.2. The results indicate that, for a 9 inch transverse crack spacing, the deflection at the edge should exceed approximately 1.6 times the uncracked edge deflections for the interior transverse stress to exceed the uncracked edge stress. This deflection ratio is possible only when serious loss of load transfer at the cracks occurs. Taute et al concluded that this condition is likely only when same degree of loss of subgrade (or subbase) support is present, i.e., voids underneath the slab should exist. These results do not have much practical value for field evaluation purposes since the edge deflection prior to cracking will not be known. The investigation of the effect of voids underneath the slab on stresses in the slab seems to be the appropriate approach to follow for practical field evaluations.

Influence of Loss of Subgrade (or Subbase) Support on Stresses

The results of the study by Taute et al referred to above indicated that some loss of subgrade support is required for significant differential movements at the cracks in the slab to occur. These movements will, in time,

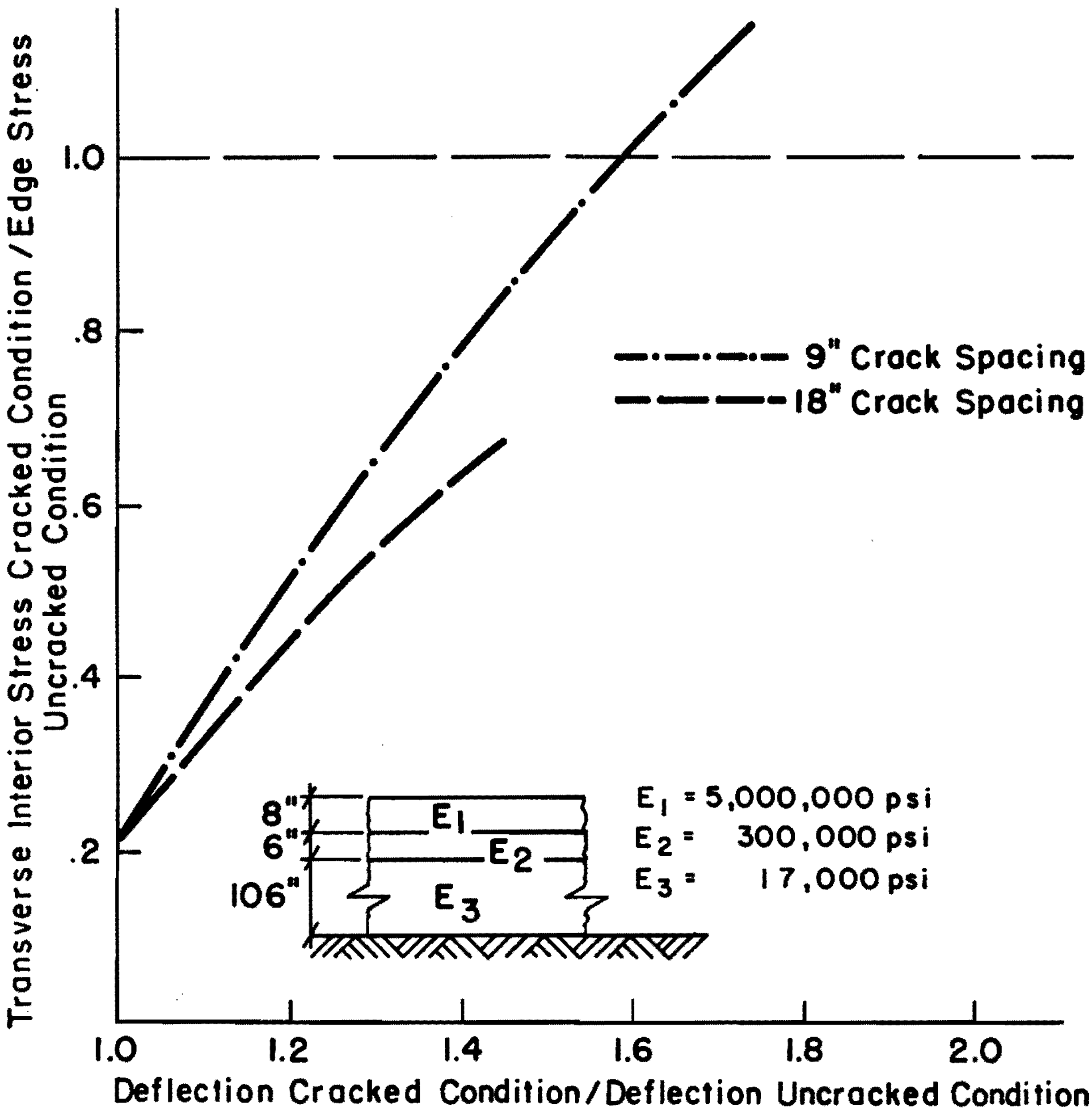


Fig 4.2. The effect of load transfer and crack spacing on the critical tensile stresses in rigid pavement for an edge loading condition (Subgrade modulus = 17,000 psi) Ref 2.

abrade the concrete at the cracks and reduce the load transfer. The reduced load transfer will result in higher deflection, at both cracks and pavement edges. The higher stresses that result will finally cause punchouts. Torres et al (Ref 6) investigated procedures for void detection and evaluated the grouting process to fill these voids. In the final evaluation of the grouting operation, they assumed that the percentage improvement in deflection due to the grouting operation is directly proportional to the resulting decrease in stresses. A procedure to evaluate the effectiveness of the grouting operation which can estimate the percent of void area filled was developed in that study.

Selection of Critical Stress Factors

A critical stress factor is necessary for adjusting the stresses calculated for the interior midspan condition for the influence of discontinuities on the critical response. Using finite element theory, Seeds et al (Ref 4) developed ranges of critical stress factors for various combinations of existing pavement-overlay-shoulder combinations. The results of this study have been incorporated in the design manual in the form of four tables:

(1) a table for the selection of the critical stress factors of the existing pavement for different shoulder types,

(2) a table for the selection of critical stress factors for various existing pavement-overlay-shoulder combinations when no significant evidence of loss of load transfer (at cracks) and voids are present,

(3) a table for the selection of critical stress factors for various existing pavement-overlay combinations when loss of load transfer and voids are present and an attempt was made to fill the voids, and

(4) a table presenting the critical stress factors for the conditions when the existing pavement was mechanically broken up.

The first table incorporated values recommended by Seeds et al. In the second table the lower limits of the range of values recommended by Seeds et al were used. The third table allows for interpolations in the range of values recommended by Seeds et al as a fraction of the percentage of the void filled by the grouting operation. In the last table, intermediate values for critical stress factors were used in the absence of better information.

The design manual provides a procedure for using deflection and condition survey information to select an appropriate repair strategy. The effectiveness of the repair strategy is finally reflected in the form of the critical stress factor selected for use in the design model. It is clear that more research is necessary to improve the guidelines for the selection of the critical stress factor as a function of the condition of the existing pavement and the repair strategy used.

CHAPTER 5. DEVELOPMENT OF DESIGN CHARTS

In order to determine the fatigue life of an overlay, it is necessary to predict the critical pavement response. The Texas Rigid Pavement Overlay Design System uses elastic layer theory to predict the response for the interior loading conditions. A critical stress factor, developed from finite element theory, is then used to adjust the interior response to the critical response. The critical response is then entered into an appropriate fatigue equation to obtain the fatigue life of the layer considered. In order to provide charts for the approximate hand solutions for the design of typical overlays, it was necessary to incorporate (1) the determination of interior response, (2) the critical stress factors, and (3) the appropriate fatigue equation into each design chart.

INTERIOR RESPONSE

A Layer Regression Submodel (REGRSP) form part of the computer program, RPRDS1, developed by Seeds et al (Ref 4). This submodel contains a set of 12 regression equations for the calculation of interior pavement response. The development of these equations together with discussions of their prediction accuracy, is well documented by Seeds et al (Ref 4). They designed an experiment where the significant factors (the independent variables, such as E-moduli and layer thicknesses) which affect the response (dependent variable) predicted by elastic layer theory could be varied to produce a factorial of elastic layer solutions. Less significant factors, such as Poisson's ratio, were fixed. A log ($base_{10}$) transformation of the response (dependent variable) was used and a stepwise regression performed to determine the equations. The accuracy of the equations is generally in the order of 95 percent. It was necessary to use four of these equations to construct charts capable of predicting the interior responses required to design typical overlays in Texas. These are

(1) the E21 regression model for predicting the tensile strain in AC overlays;

(2) the S32 regression model for predicting the stress in the PCC slab of a 3-layer concrete pavement;

(3) the 5B42 regression model for predicting the stress in the original PCC slab after an AC or PCC overlay have been constructed; and

(4) the SU51C regression model for predicting the stress in an unbonded PCC overlay of a 5-layer concrete pavement (original PCC slab cracked).

Significant information about the constraints on each of these equations and plots of the accuracy of each equation as well as the regression equations themselves are presented in Appendix C.

CONSTRUCTION OF DESIGN CHARTS

The multi-factor regression equations selected for the construction of the design charts contains up to 20 terms. Basic nomographic theory was used to combine these terms in the charts. It was however not feasible to incorporate all the terms of the equations in the charts. The technique used to get around this problem was to do a sensitivity analysis to identify the significant terms of the equation. These terms are incorporated directly into the charts. The less significant terms are incorporated as average values in the first phase of each design chart. The other phases of each chart are then used to do corrections around the average values assumed in the first phase of each chart. Using this technique, up to four corrections could be applied to the less important terms, reducing the errors (caused by assuming average values for these terms) to less than the reading error of the charts. The regression equations developed by Seeds et al predict only the response to the interior loading condition. The last phase of each design chart, however incorporates the appropriate fatigue equations and the selection of a critical stress factor to enable the design engineer to use the charts to predict pavement life directly. The fatigue equations used in the charts are shown in Figs 5.1 and 5.2.

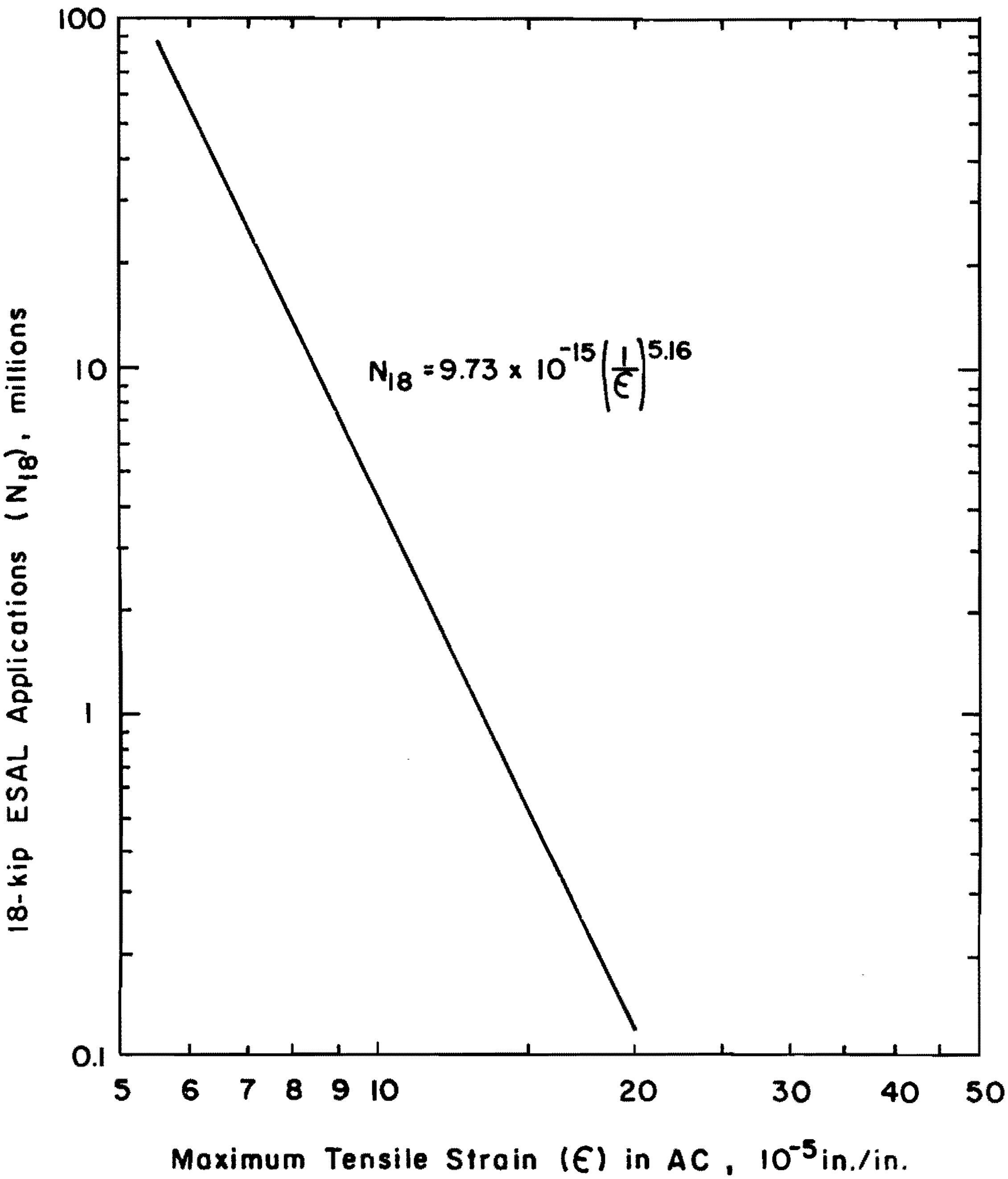


Fig 5.1. AC fatigue equation (Ref 4).

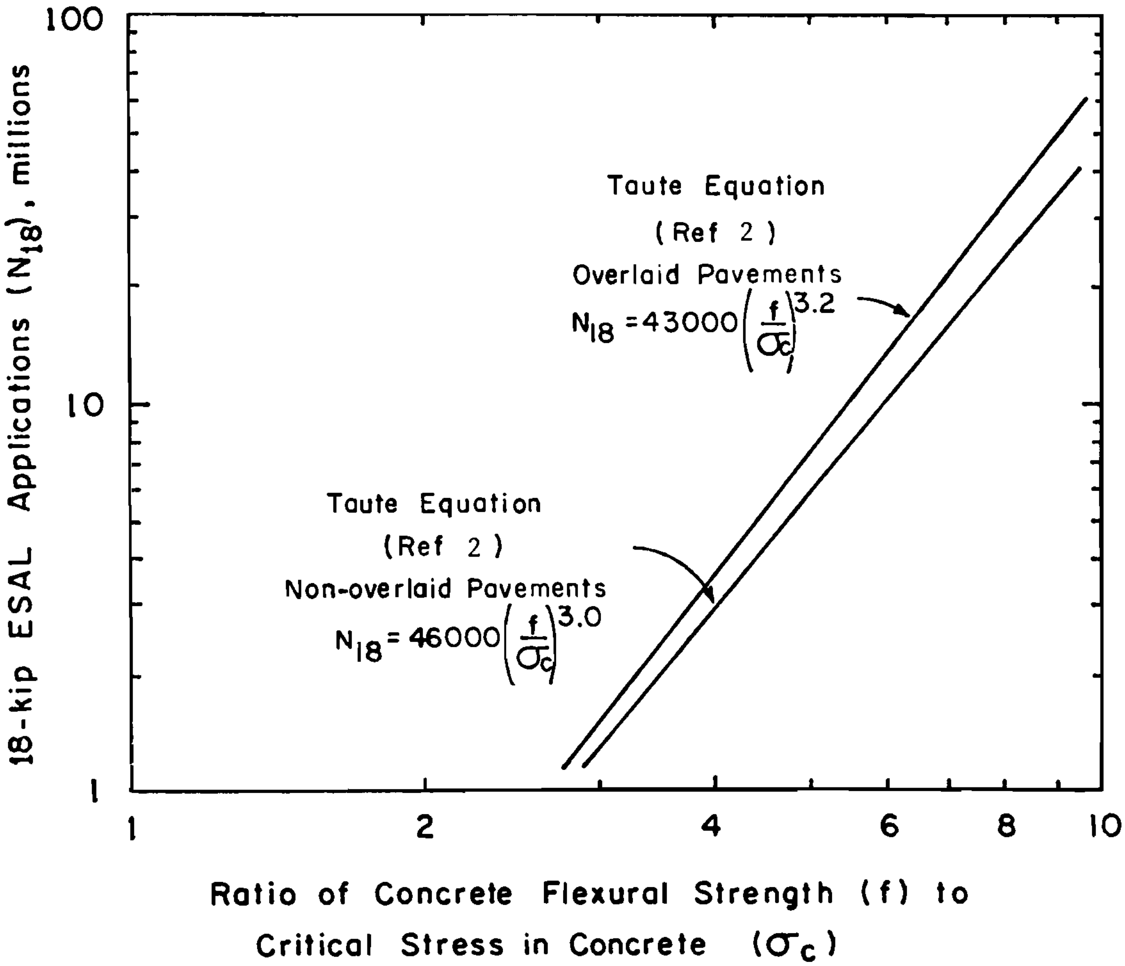


Fig 5.2. PCC fatigue equations.

APPLICATION OF DESIGN CHARTS

The four design charts presented in Section A12 can be used to determine the required thickness of the first overlay on an existing rigid pavement consisting of three layers (i.e., PCC slab, subbase, and subgrade). AC overlays and both bonded and unbonded PCC overlays can be designed. Since one of the charts Fig A12.3) in Section A12 of the design manual predict the stress and fatigue life of the original PCC slab. This chart can also be used to determine the original fatigue life of the existing PCC slab. The charts predict 100 percent fatigue life of the layer considered. When the remaining fatigue life of the layer is less than 100 percent, adjustments should be made according to the design philosophy adopted in computer program RPRDS-1 (Ref 4). This design philosophy is summarized in Section A12 of the design manual and a flow diagram shows a step-by-step procedure for making the necessary adjustments.

LIMITATION OF DESIGN CHARTS

Geometric Limitations. The following limitations are presented:

(1) Only 3-layer original pavement structures can be considered, and

(2) the range of layer thicknesses and material properties incorporated in the charts are limited.

ACCURACY

The accuracy of the regression equations used to develop the charts, are in the order of 95 percent. Even though this error in the response value can result in fairly large differences in predicted fatigue life when compared to the lives calculated using elastic layer theory; it will general not have a significant affect on the overlay thickness predicted. The accuracy of the regression equations (and therefore also the design charts) is more than enough, however, for developing approximate hand solutions for overlay thickness design problems.

CHAPTER 6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter presents a summary of the findings of this study. The additions to the Texas Rigid Pavement Overlay Design Procedure, as well as the main features of the Design Manual presented in Appendix A of this report, are discussed. Finally, the principle conclusions of this study and recommendations for further research are provided.

SUMMARY

A Rigid Pavement Overlay Design Manual has been compiled. This manual can be used to obtain approximate hand solutions to overlay design problems, as well as to prepare design inputs to more precise automated design models. The background and development of procedures incorporated in the manual are described in the main body of the report, and the design manual is presented in Appendix A.

The design manual can be broadly classified into four phases. The first phase deals with the collection and reduction of design information. The second phase describes how to use design information to determine designer inputs. The design charts and design philosophy for overlay thickness design are described in the third phase. The last phase shows a simplified procedure to do a net present worth of cost analysis. Finally, a design example is provided.

Significant additions to the Texas Rigid Pavement Overlay Design Procedure are presented in phases 2 and 3. These include

(1) A nomograph for predicting remaining life from condition survey data. This estimate of remaining life is then compared with the remaining life of the fatigue model and techniques on how to use this comparison to improve the inputs to the design procedure are provided.

(2) A set of tables for determining the optimal design strategies for different traffic and subgrade support conditions. These tables can also be used to get an indication of the consequences (in terms of relative costs) of not implementing the optimal strategy. Finally an estimate of user delay cost as a percentage of total cost is presented in the tables.

(3) Four design charts for overlay thickness design. The charts can be used for predicting the pavement response and to determine the design life of typical overlay design strategies.

CONCLUSIONS

(1) The rigid pavement manual presented in this report is based on the Texas Rigid Pavement Overlay Design Procedure. The techniques and procedures outlined in the design manual can therefore also be used to define design inputs for the more precise automated design models incorporated in the Texas Rigid Pavement Overlay Design System.

(2) A set of tables has been developed for the selection of optimal design strategies. These tables were developed using computer program RPRDS-1 with typical design inputs from Texas. The general guidelines which can be obtained from the tables, may however be usual for wider application since they have been developed for a wide range of traffic and subgrade support conditions.

(3) The charts for the design of overlay thickness are accurate reflection of multi-factor regression equations (based on layer theory calculations of pavement response), combined with appropriate fatigue equations. Their accuracy however, is dependent on the accuracy of the regression equations and their application is limited to the design of the first overlay to typical 3-layer original pavement structures.

RECOMMENDATIONS

(1) It is recommended that detailed research be conducted to develop a relationship between distress occurrence and traffic applications which considers the stress produced by loads. Such a study is needed to improve the estimation of remaining life and it's application in the material characterization procedure.

(2) Improved procedures should be developed to consider a repair strategy in the design model, in particular the influence of a repair strategy on the critical stresses in the original PCC slab and the overlays.

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APPENDIX A

THE RIGID PAVEMENT OVERLAY DESIGN MANUAL

SECTION A1: THE DESIGN PROCEDURE

SCOPE

The overlay design procedure presented in this manual encompasses the design of both asphaltic concrete (AC) and portland cement concrete (PCC) overlays on existing rigid pavement structures. The manual presents a simplified procedure based on the comprehensive rigid pavement overlay design system developed as part of a cooperative research program between the State Department of Highways and Public Transportation (SDHPT) and the Center for Transportation Research (CTR) at the The University of Texas at Austin. The procedure is intended for approximate hand solutions to overlay design projects. The design philosophy and the basic steps in the simplified procedure are identical to those of the comprehensive design system. Techniques and design tables presented in this manual will thus be very useful in the preparation of design inputs for automated solutions. The segments of the procedure that can be automated are clearly delineated in the procedure and the appropriate references that are supplied.

BASIS OF THE MANUAL

A combination of elastic layer theory and finite element theory is used to predict the critical pavement response (i.e., tensile stress or strain) required in estimating the life of a given overlay. Elastic layer theory is first used to predict the pavement response for the interior condition (away from an edge, corner, or crack). Appropriate critical stress factors, based on finite element theory, are then used to derive the critical response from the interior response. Two distress mechanisms are modelled for overlay thickness design:

(1) Fatigue cracking mechanism. The basic overlay thickness is obtained using the critical stress (PCC layer) or strain (AC layer)

in an appropriate fatigue equation to determine a structurally balanced overlay thickness.

(2) Reflection cracking mechanism. The thickness obtained in (1) is checked for reflection cracking. Two reflection cracking mechanisms are considered, (a) horizontal movements of the underlying slab (as a result of temperature and moisture changes) and (b) traffic induced differential vertical movements across pavement discontinuities (i.e., cracks or joints) in the original pavement.

Finally, in order for the designer to make a fair comparison between alternative design strategies, a present worth of cost analysis is used as a common basis for comparing alternative design strategies.

STEPS IN THE DESIGN PROCEDURE

Figure A1.1 illustrates the basic steps in the design procedure and serves as an outline for the description of the procedure in the rest of the manual. Steps which can be fully or partially automated are also indicated in Fig A1.1. Table A1.1 serves as a supporting table to Fig A1.1 and summarizes the capabilities of the available computer programs. The next three sections (Section A2, A3 and A4) deal with the collection and reduction of design information. Sections A5 to A11 describe how design information can be used to arrive at design inputs. The recommended procedures for overlay thickness design are presented in Section A12. Section A13 deals with the cost analysis. Finally, a design example is presented in Section A14.

SECTION A2: CONDITION SURVEY

Condition survey information is not used directly in the design procedure. This information is, however, reflected indirectly in various important design inputs, such as the (a) delineation of design sections, (b)

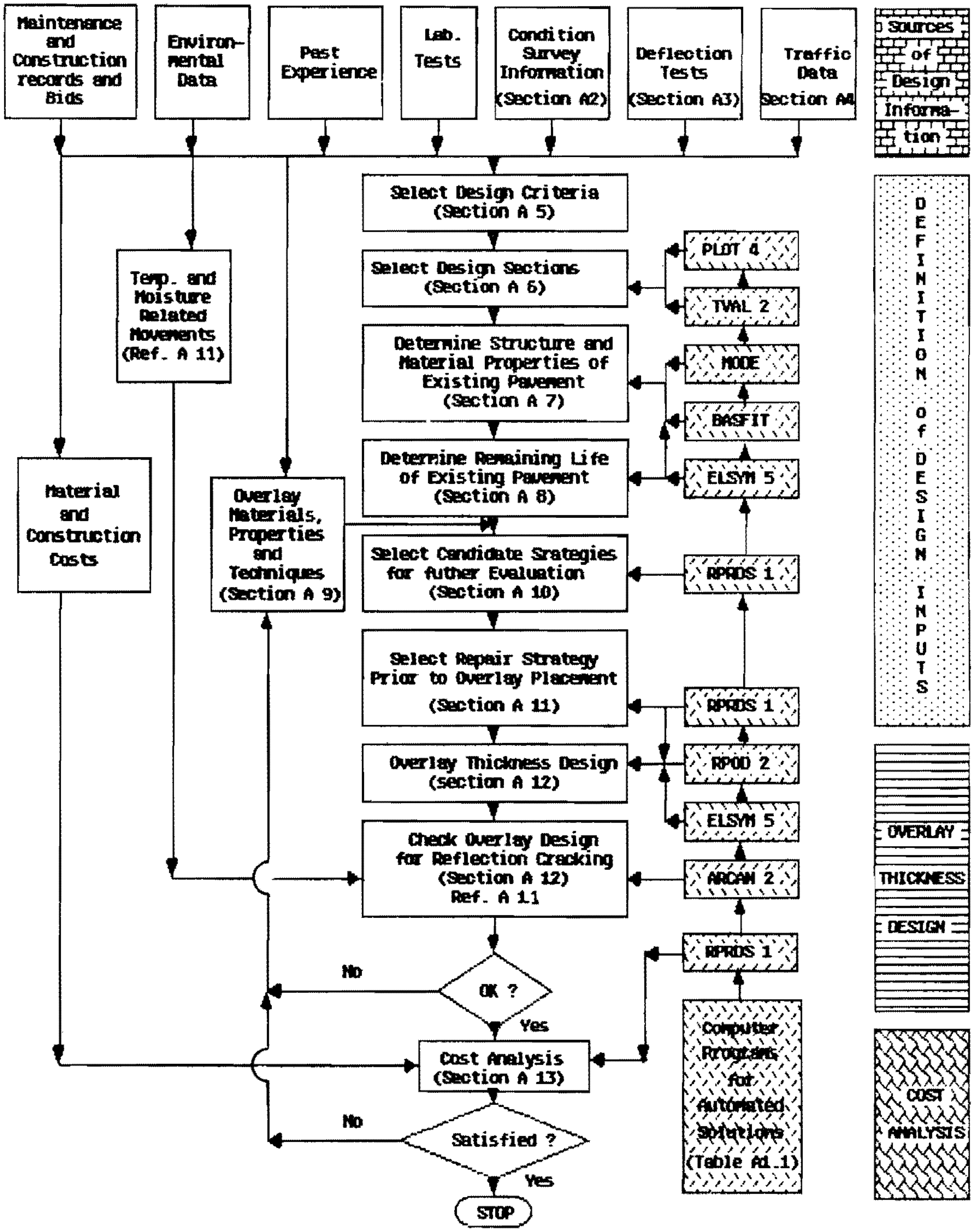


Fig A1.1. The Rigid Pavement Overlay Design Procedure.

TABLE A1.1. AVAILABLE COMPUTER PROGRAMS WHICH CAN BE USED IN THE DESIGN PROCEDURE

Program	Description	Reference
PLOT 4	Plots deflection profiles	A1
TVAL 2	Test sections for statistically significant differences in terms of deflection	A1
MODE	Plots the frequency and cumulative distributions of a number of design parameters calculated from Dynaflect deflection measurements	A2
BASFIT	Interative version of elastic layer program for back-calculation of Young's moduli from deflection basin	A3
ELSYM 5	Elastic layer program calculating pavement response (deflection, stress or strain). Can also be used to fit theoretical deflection basin to measured deflection basin for back-calculation of Young's moduli	A4
RPRDS-1	Generates, analyses (using fatigue cracking mechanism) and compares numberous overlay design strategies on a cost basis	A5
RPOD 2	Calculates the relationship between overlay thickness and fatigue life for a specific design strategy	A1
ARCAN 2	Determine the extent of reflection cracking of an AC overlay as a function of time and traffic	A6

estimation of the remaining life of the existing pavement, and (c) selection of design and maintenance and repair strategies. Condition survey procedures and forms are discussed in depth in reference A7 and A8.

CONDITION SURVEY FORMS

Because of the differences in distress manifestations characteristic to the two basic types of rigid pavements, slightly different procedures and forms have been developed for CRC and JC pavements respectively.

CRC Pavements

The basic CRCP recording form is presented in Fig A2.1. The distress manifestations accommodated by the form together with a short description of each distress manifestation and a recommended recording procedure are summarized below.

Minor and Severely Spalled Cracks. Spalling is defined as the widening of existing cracks by secondary cracking or breaking of the crack edges. The depth of a spall is generally less than one inch, but it can be very wide. Minor and severely spalled cracks are distinguished by the width of the spall.

Minor spalling is defined as a condition of edge cracking in which the loss of material has resulted in a spall of roughly one-half inch in width. Severe spalling is defined as a condition in which the spall is wider than one-half inch.

Separate counts are made of the total number of transverse cracks, and cracks showing signs of spalling. The condition of the whole crack is defined by the most severe condition of spalling along that crack.

Punchouts. When closely spaced transverse cracks are linked by longitudinal cracks to form a block, the block is called a punchout. This must not be confused with longitudinal cracking, which is not recorded on the sheet. A minor punchout is defined as a condition where, although a block has formed, no sign of movement under the traffic is apparent. The cracks

CRC PAVEMENT CONDITION SURVEY FORM FOR SMALL SECTIONS

DISTRICT	CONTROL	SECTION	JOB Nº	CFTR #	HIGHWAY AND DIRECTION	COUNTY	DATE	

LOCATIONS		COMMENTS	DATE OF SURVEY	NUMBER OF CRACKS	NUMBER OF SPALLED CRKS		PUNCHOUTS				PATCHES		PUMPING	
							MINOR		SEVERE					
FROM	TO				MINOR	SEVERE	< 20'	> 20'	< 20'	> 20'	A.C.	PCC	MINOR	SEVERE
1–8	9–16	17–33	34–40	41–46	47–50	51–54	55–57	58–60	61–63	64–66	67–69	70–72	73–76	77–80

Fig A2.1. CRCP condition survey form for small sections (Ref A8).

surrounding the punchout are narrow and few signs of spalling are apparent. A severe punchout is recorded when the block moves under traffic. The surrounding cracks will be fairly wide and signs of pumping around the edge of the block may be apparent. Punchouts are divided into two categories: those shorter than 20 feet and those longer than 20 feet. The length of a punchout is determined by the length of the longitudinal crack forming a side of the punchout. Even if this longitudinal crack were to extend across several transverse cracks, only one punchout would be recorded.

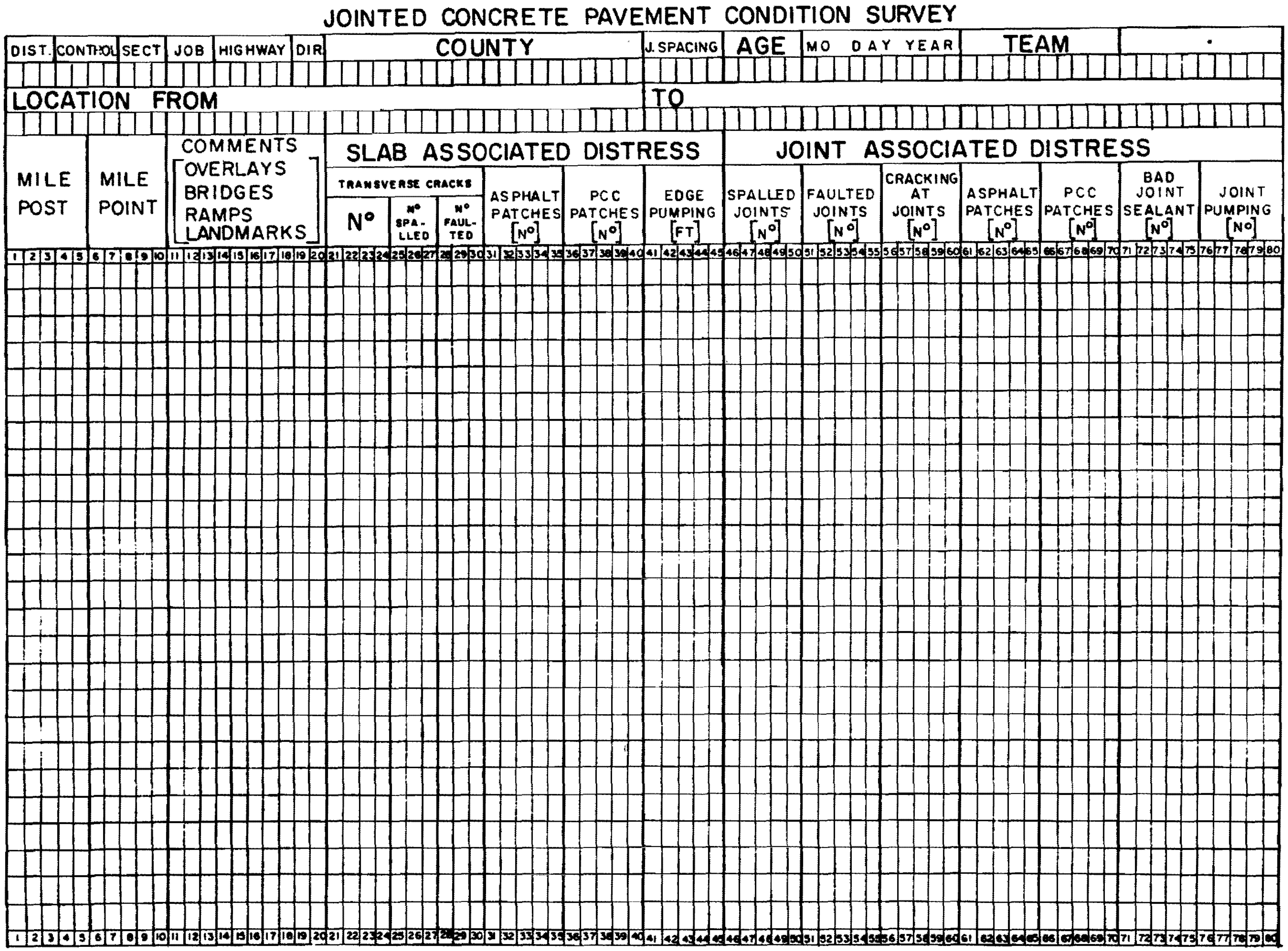
Repair Patches. Severe punchouts are repaired by patching the pavement. A repair patch is defined as a repair section of the pavement where the repair work has been carried out to the full depth of the concrete. Asphaltic concrete repair patches and portland cement concrete repair patches are distinguished from each other. The number and condition of the patches in the section should be recorded.

Pumping. Water passes through cracks and openings in the pavement and penetrates the sublayers. When a load, such as a heavy vehicle passing over a crack, is applied, the water is pressed out of the crack, taking fine material of the sublayers with it. This is defined as pumping. Pumping may occur at transverse and longitudinal cracks and construction joints. Minor pumping has occurred when water pumped out, leaves streaks of fines on the pavement surface. Severe pumping indicates a severe loss of fines from sublayers and may also be associated with permanent vertical displacement of the pavement. Sections which exhibit pumping as well as the severity of the pumping must be recorded.

JC and JRC Pavements

The basic JCP and JRCP recording form is presented in Fig A2.2. The distress manifestations accommodated by the form together with a short description of each manifestation and recommended recording procedure are summarized below.

Slab Associated Distress. These distress manifestations occur along the length of the slab and not in the vicinity of a joint. The first three distress manifestations relate only to jointed reinforced concrete pavements.


JOINTED CONCRETE PAVEMENT CONDITION SURVEY

DIST. | CONTROL | SECT | JOB | HIGHWAY | DIR | COUNTY | J. SPACING | AGE | MO DAY YEAR | TEAM

LOCATION FROM | TO

MILE POST	MILE POINT	COMMENTS [OVERLAYS BRIDGES RAMPS LANDMARKS]	SLAB ASSOCIATED DISTRESS						JOINT ASSOCIATED DISTRESS							
			TRANSVERSE CRACKS			ASPHALT PATCHES [N°]	PCC PATCHES [N°]	EDGE PUMPING [FT]	SPALLED JOINTS [N°]	FAULTED JOINTS [N°]	CRACKING AT JOINTS [N°]	ASPHALT PATCHES [N°]	PCC PATCHES [N°]	BAD JOINT SEALANT [N°]	JOINT PUMPING [N°]
			N°	N° SPALLED	N° FAULTED										
1–5	6–10	11–20	21–24	25–27	28–30	31–35	36–40	41–45	46–50	51–55	56–60	61–65	66–70	71–75	76–80

Fig A2.2. Field sheet for recording distress of jointed concrete pavements (Ref A8).

(a) Transverse cracks. Transverse cracks occur at intervals along the slab. Transverse cracks in the vicinity of a joint, which may have resulted from some joint defect, do not fall into this category. Transverse cracks occur as a result of temperature drop stresses, drying shrinkage, and traffic loading.

(b) Spalled transverse cracks. Spalling is the widening of existing cracks by secondary cracking or breaking of the concrete at the edges. Spalling results from traffic loading and from stresses which occur because of material which enters the crack and resists thermal expansion. Both these situations result in high stresses in he upper edge of the concrete along the crack, and a spall results.

The number of spalled cracks in the outer lane is recorded. If the spall is less than an inch in width and depth and only a few of these spalls occur along the length of a crack, the crack is not counted as spalled. For a crack to be counted as spalled, a significant amount of spalling must have occurred and a drop in the riding quality of the pavement must result. If the spall has been patched, the spalled crack should be counted, not the patch.

(c) Faulted transverse cracks. Faulted transverse cracks occur as a result of a loss in subgrade support and traffic loading. The concrete in the immediate vicinity of the steel will break off and the final result will be the difference in the level of the slab on either side of the crack. This will result in a significant loss of riding quality.

The number of faulted transverse cracks in the outer lane of the section in recorded.

(d) Slab patches. The number of repair patches in both lanes of the roadway is recorded. Portland cement concrete and asphalt concrete patches are recorded separately. Neither the condition nor the size of the patch is recorded.

(e) Edge pumping. Water passes through cracks in the pavement and penetrates the sublayers. When a load, such as a heavy vehicle passes over the crack, the water is forced out of the crack, taking fine material of the sublayers with it. This is defined as pumping. Pumping generally leaves a stain on the shoulder of the road, and hence is easily noticed from inside the survey vehicle.

The length of the edge crack causing this staining is estimated and divided by the length of the section (approximately 1,000 feet) to arrive at a percentage. Because it is difficult to estimate the length of the edge crack which is pumping, this result will be slightly subjective.

Joint Associated Distress. This distress should be directly related to the joints in the pavement.

(a) Spalled joints. Spalled joints occur in a manner similar to the occurrence of spalled cracks. The number of joints exhibiting spalls which are wider and deeper than one inch is recorded. The whole joint across both trafficked lanes should be examined for spalls.

CONDITION SURVEY PROCEDURE

The condition of the existing pavement should be carefully documented at project level. The persons making the survey should preferably walk along the side of the road. A measuring tape can be used to determine the distance between distress manifestations. The uniformity of the section will determine the length of base elements and station limits, normally 100 feet long, form useful starting lengths. A sheet of opaque plastic can be used to sketch the nature and location of distress manifestations for accurate recording. This process is especially useful when special deflection studies are required (refer to Sections A3 and A11) to supplement the deflection data with the accurate recording of distress manifestations.

DATA REDUCTION

In order to use the semi empirical relationships between the present condition and the remaining life of the existing pavement (as presented in Section A8), it is recommended that a distress index (Z-value) be used to combine distress manifestations to ascertain with a single number, the amount of pavement deterioration. Two different equations are used for CRCP and JCP respectively (Ref A8).

Z-value for CRCP (Z_c)

$$Z_c = 1.0 - 0.065\ FF - 0.015\ MS - 0.009\ SS$$

where

FF = number of failures per mile, i.e., sum of punchouts and patches

MS = percent minor spalling

SS = percent severe spalling

Z-value for JCP and JRCP (Z_j)

$$Z_j = 1.0 - 0.005\ CRK - 0.006\ PS - 0.003\ FLT$$

where

CRK = number of cracks per mile

PS = percent spalled joints and cracks

FF = number of faulted joints and cracks per mile

SECTION A3: COLLECTION AND REDUCTION OF DYNAFLECT DEFLECTION DATA

DEFLECTION APPARATUS

The Dynaflect is a popular non destructive testing equipment currently in use by different agencies in the USA. The Dynaflect is a trailer mounted unit which induces a steady state vibratory force on the surface of the pavement through two rubber coated wheels. The dynamic force generator employs two counter rotating eccentric masses producing a peak to peak dynamic load of 1,000 lbs at a fixed frequency of 8 H_Z. Five equally spaced geophones are used to measure the deflection response of the pavement. The arrangement of five geophones in the automated system of the Dynaflect provides half of the so called deflection basin (Fig A3.1). Reference A3 provides a user manual for the Dynaflect testing of rigid pavements.

APPLICATION OF DEFLECTION DATA

In the context of rehabilitation design, the purpose of deflection measurements can be any one or a combination of the following:

Selection of Design Sections

The Dynaflect sensor_5 deflection (W_5) and deflection slope (W_1-W_5) are used to select contiguous design sections (refer to Section A6).

Material Characterization

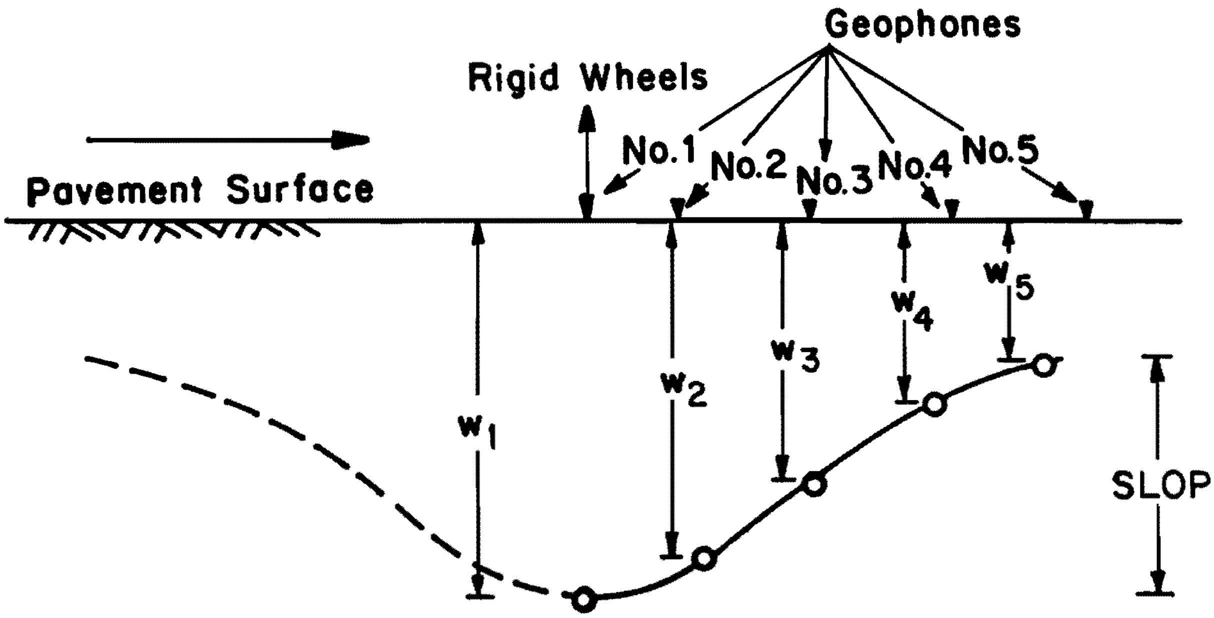
The deflection data are used for determining a design deflection value for each design section. Additionally the deflection basins can be used to back calculate in situ Young's moduli of the pavement layers (refer to Section A7).

Void Detection and Effectiveness of Grouting

The Dynaflect deflection data are used to detect the pavement areas having voids under the concrete slab. The deflection measurements taken after grouting work can also provide an estimate of the effectiveness of the under sealing work (Section A11).

Load Transfer

Deflection data are used to evaluate the structural condition of a discontinuity to estimate the loss of load transfer across the transverse discontinuities on rigid pavements (Section A11).



Maximum Dynaflect Deflection = w_1

Basin Slope, SLOP = $w_1 - w_5$

Fig A3.1. Typical Dynaflect deflection basin.

PLANNING OF DEFLECTION MEASUREMENTS

When deflection measurements are planned, aspects such as the required number of tests, test location, time of testing and the recording and reduction of data should be considered. These aspects are a function of the purpose of the measurement. Figure A3.2 presents an outline of this process.

Selection of Design Sections and Material Characterization

Deflection Parameters Required. The W_5 deflection and deflection slope W_1-W_5 are the deflection parameters used for the selection of design sections and for material characterization.

Location of Deflection Tests. The recommended location is 6 feet from the pavement edge in the outside lane (midspan position) as illustrated in Fig A3.3.

Required Number of Deflection Tests. In order to determine the required number of deflection tests, the length of the design section must be known. An initial length of 1,000 feet can, however, be used to obtain an estimate of this number. The procedure described below can be used to check of the number of tests used were sufficient for the selected allowable error and confidence level.

Use the following equation to compute the size of the population of deflection measurements, N.

$$N = \frac{L}{\bar{S}}$$

where

L = pavement section length, feet

$\bar{S}$ = average spacing between successive discontinuities in the longitudinal direction, feet.

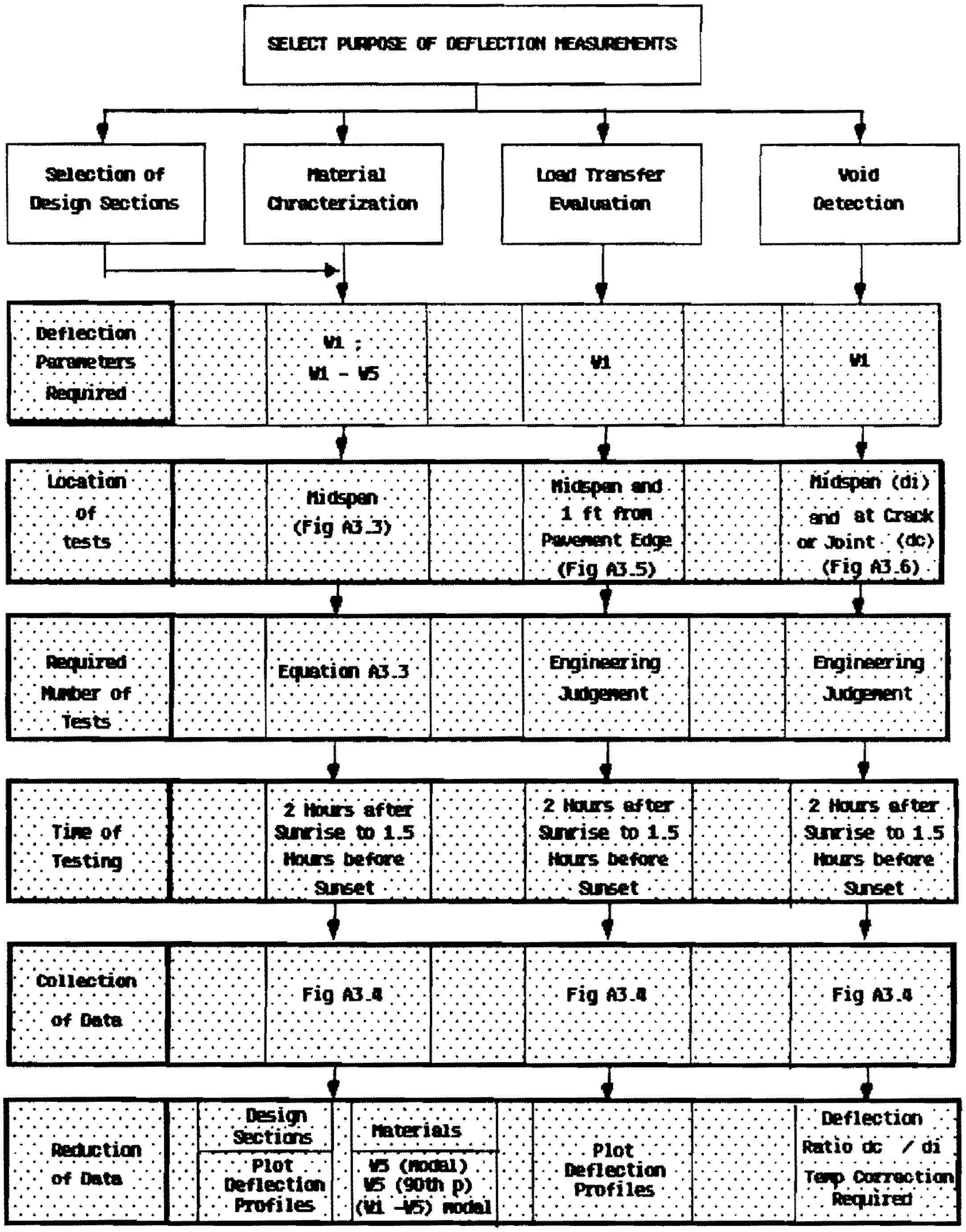
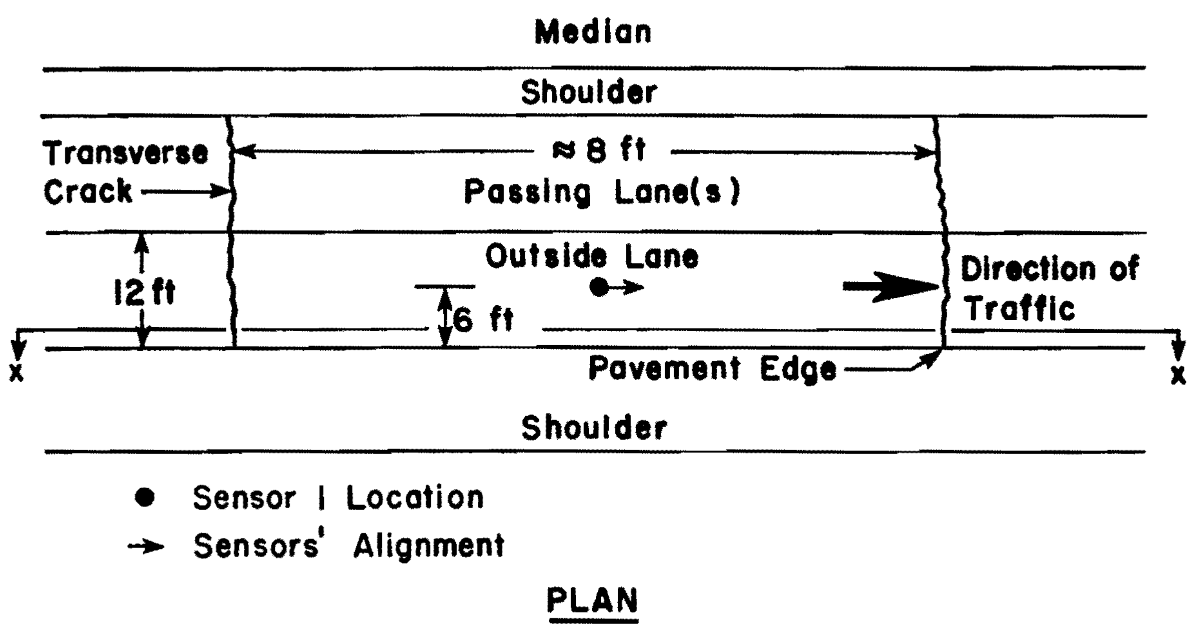


Fig A3.2. Deflection testing needs for rehabilitation design.



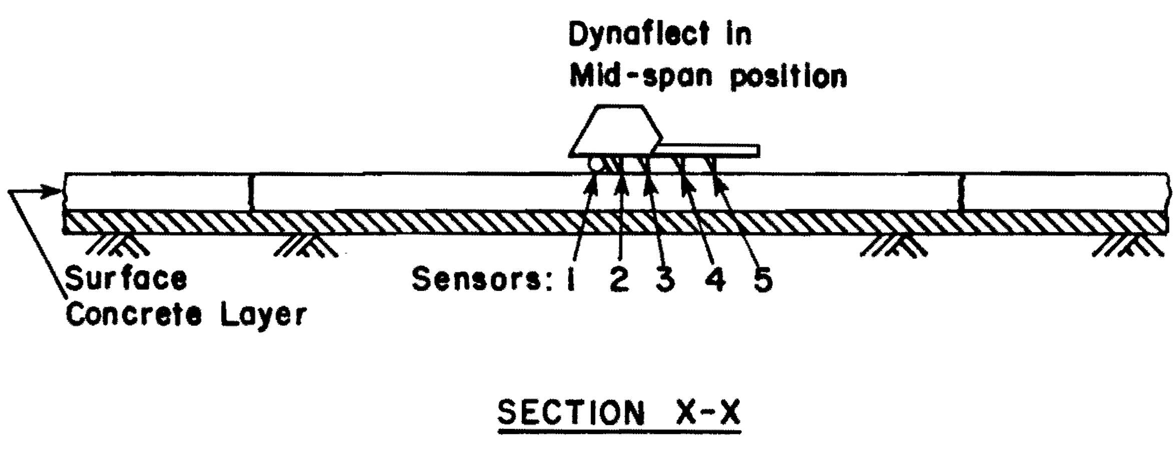


Fig A3.3. Recommended test locations for material characterization on CRC pavement of a typical divided highway (Ref A3).

A minimum section length of 1,000 feet is recommended. $\bar{S}$ can be obtained from condition survey data corresponding to average crack spacing for CRCP, and average joint spacing for JRCP or JCP.

Compute $\hat{\sigma}$ the estimator of the standard deviation of the population of required deflection parameter by means of Eq A3.2 (Ref A3).

$$\sigma = \frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n - 1} \qquad \text{(A3.2)}$$

where

x_i = value of the sample's i^{th} sensor 1 deflection, mils

n = sample size

$\bar{x}$ = sample mean

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i$$

Select an allowable error, e, expressed as a function of sensor 1 mean deflection. E can be related to variation in slab thickness if the guidelines provided in Table A3.1 are followed.

Obtain Z_α from Table A3.2 according to the desired confidence level. A confidence level of 90 or 95 percent is commonly selected.

Determine the required number of Dynaflect deflections using Eq A3.3 (Ref A3).

$$n_r = \frac{1}{\dfrac{e^2}{Z_\alpha^{\,2}\,\sigma^2} + \dfrac{1}{N}} \qquad \text{(A.3.3)}$$

<u>Time of Testing</u>. Commence deflection measurements two hours after sunrise and stop 1-1/2 to 2 hours before sunset. No temperature correction is required.

TABLE A3.1 RELATIONSHIP BETWEEN ALLOWABLE ERROR, e AND VARIATION IN SLAB THICKNESS (REF A3)

e, mils	Variation in slab thickness, in
0.025 $\bar{x}$	0.25
0.050 $\bar{x}$	0.50
0.100 $\bar{x}$	1.00

TABLE A3.2 VALUES OF Z_{α} FOR VARIOUS CONFIDENCE LEVELS (REF A3)

Confidence level, α, %	Z_{α}
80.0	0.842
85.0	1.036
90.0	1.282
95.0	1.645
97.5	1.960
99.0	2.326

Collection of Data. The following are recommended:

(1) Preparation of data sheets: Deflection measurements made at different test locations are to be distinguished by the inclusion of a standard abbreviation in the first three columns of the standard deflection data sheet (Fig A3.4) as suggested by Taute et al (Ref A2).

MID - midspan deflection
CRK - deflection at a crack
JNT - joint deflection
EDG - deflection near pavement edge.

The purpose of deflection measurement should also be included in the remarks columns. The suggested abbreviations are:

MC - material characterization
DV - detection of voids
US - effectiveness of grouting
LT - load transfer
RM - replicate measurement.

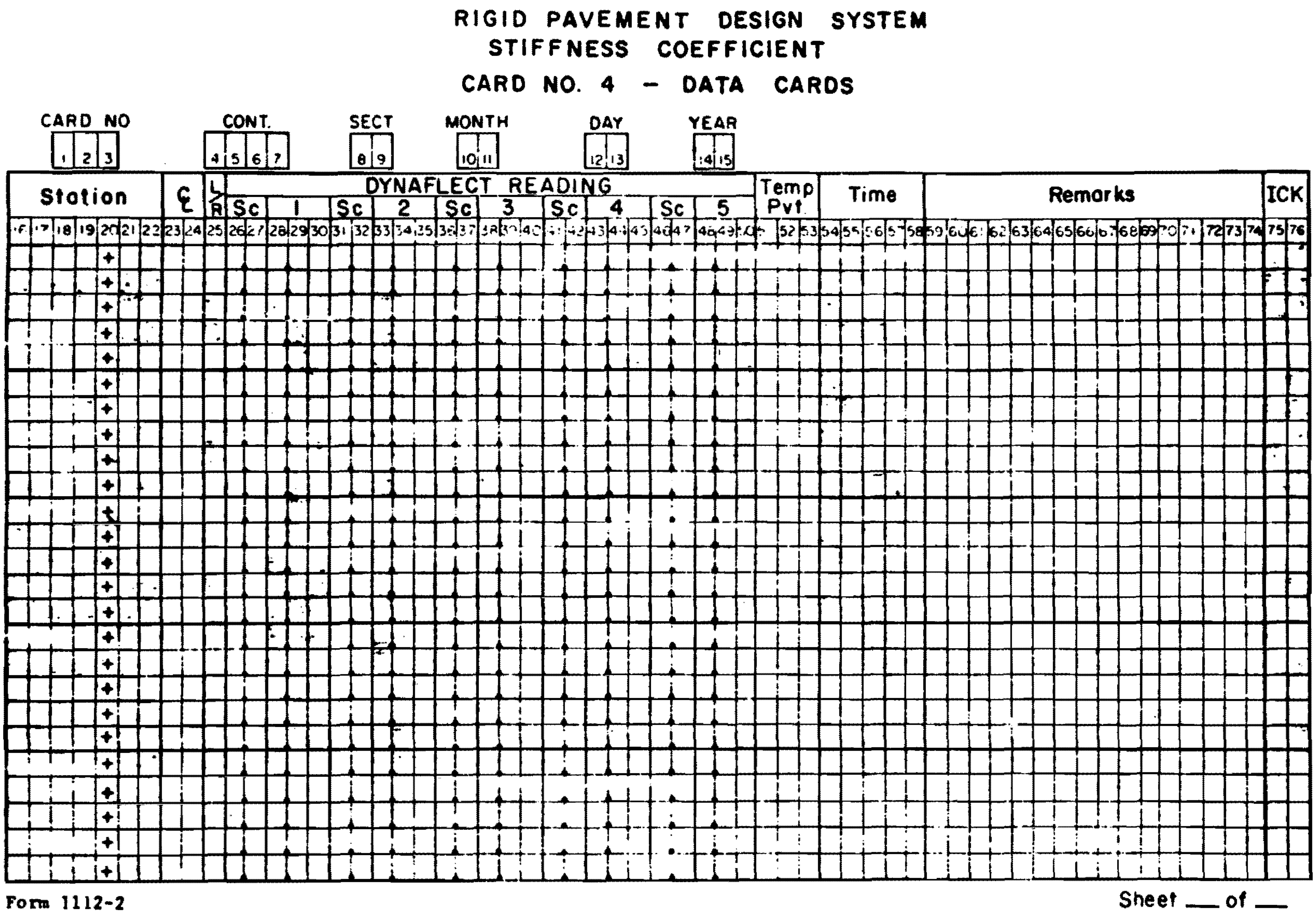
It is also important to note down the time in the appropriate columns at every instance of deflection measurement.

(2) A sketch showing the Dynaflect position on the pavement should accompany the recorded deflection data sheets. Figures A3.3, A3.5 and A3.6 can be used for this purpose.

Reduction of Data. The computer program PLOT4 (Ref A1) and MODE (Ref A1) can be used to reduce data. The profile of deflection parameters and deflection statistics can also be determined by hand.

Void Detecting and Effectiveness of Grouting

Deflection Parameters Required. Dynaflect sensor 1 (W_1) deflection are required for this purpose.



RIGID PAVEMENT DESIGN SYSTEM

STIFFNESS COEFFICIENT

CARD NO. 4 – DATA CARDS

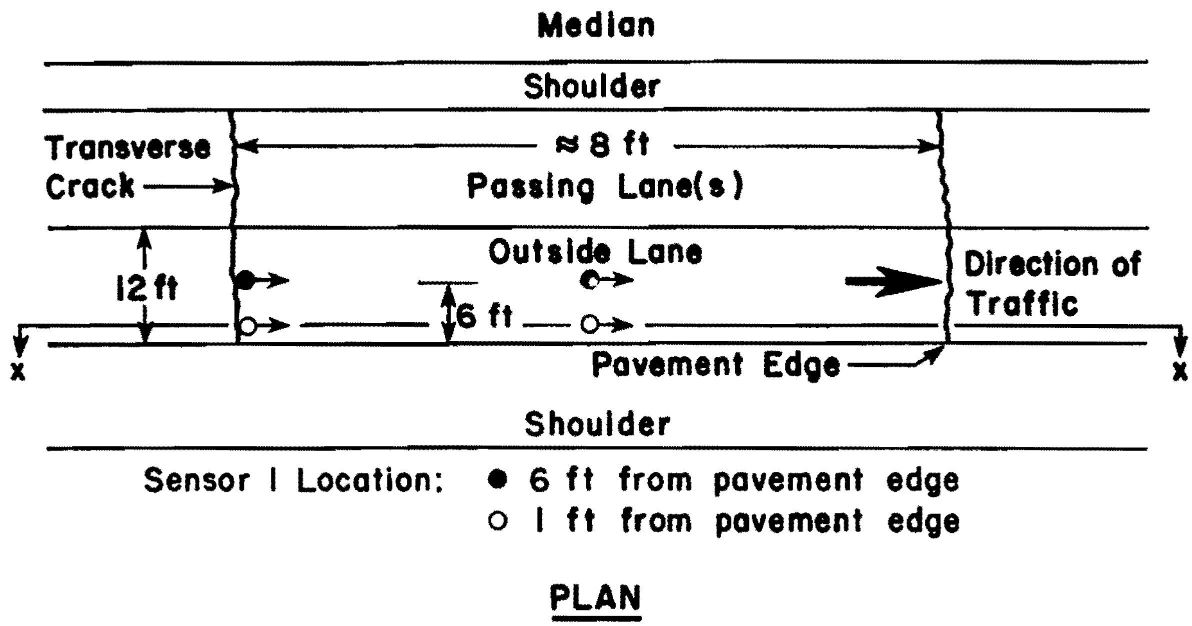
CARD NO	CONT.	SECT	MONTH	DAY	YEAR
1 2 3	4 5 6 7	8 9	10 11	12 13	14 15

Station	℄	L/R	DYNAFLECT READING Sc	1	Sc	2	Sc	3	Sc	4	Sc	5	Temp Pvt.	Time	Remarks	ICK
16–22	23 24	25	26 27	28 29 30	31 32	33 34 35	36 37	38 39 40	41 42	43 44 45	46 47	48 49 50	51 52 53	54 55 56 57 58	59–74	75 76

Form 1112-2

Sheet ___ of ___

Fig A3.4. The SDHPT standard form to record Dynaflect deflection measurements (Ref A3)



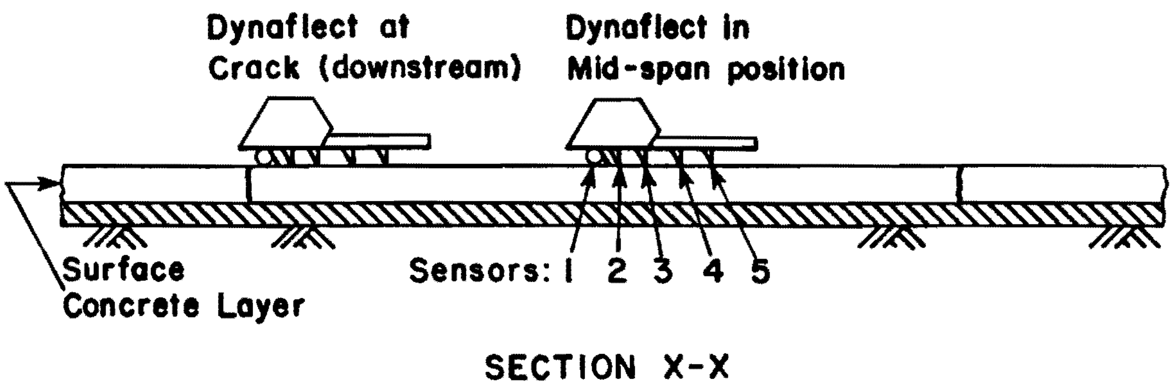


Fig A3.5. Recommended test locations for void detection on CRC pavement of a typical divided highway (Ref A3).

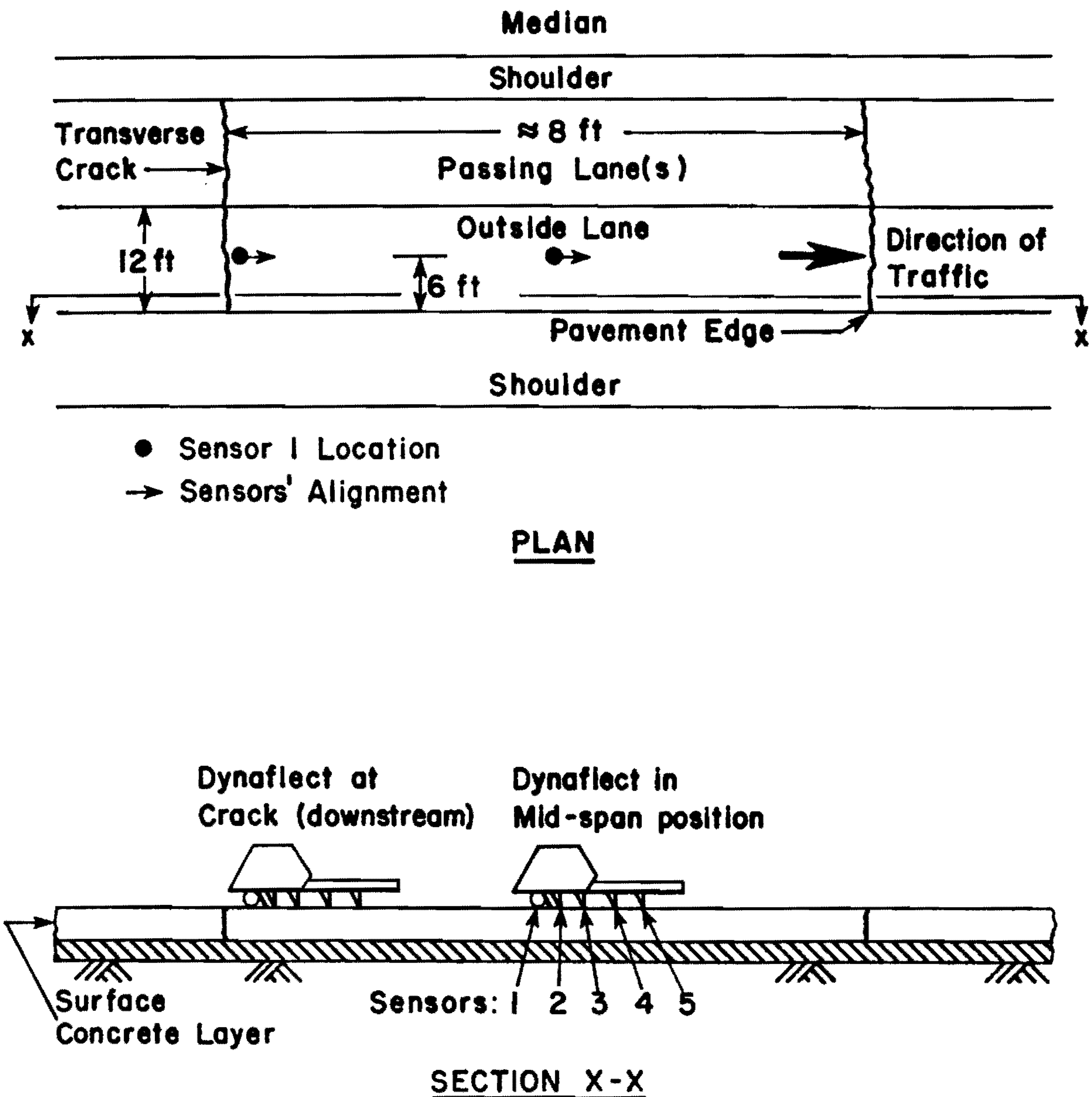


Fig A3.6. Recommended test locations for load transfer evaluation on CRC pavement of a typical divided highway. (Ref A3)

Location of Deflection Tests. Measurements at 1 feet and 6 feet from the pavement edge are required for void detection. For effectiveness of grouting evaluation, measurements at 1 ft from pavement edge are required. The test locations are illustrated in Fig A3.5.

Required Number of Tests. Engineering judgement is required. Condition survey information be used for this purpose.

Temperature Correction. Temperature corrections are required for measurements taken 1 foot from the pavement edge. The temperature correction procedure is described in Reference A3.

Collection of Data. Recommendations as given for selection of designer sections to be followed.

Reduction of Data. Plot deflection profiles as described in Section All.

Load Transfer Evaluation

Deflection Parameters Required. Sensor 1 (W_1) measurements are required for this purpose.

Location of Tests. Measurements at 6 feet from the pavement edge in both the midspan position (di) and at transverse cracks or joints (dc) are required as illustrated in Fig A3.6.

Required Number of Tests. Engineering judgement required. Condition survey information can be used for this purpose.

Temperature Correction. Temperature corrections are required for measurements taken 1 foot from pavement edge. The temperature correction procedure is described in Ref A3.

Collection of Data. Recommendations as given for selection of design sections to be followed.

Reduction of Data. The deflection ratio ($DR = \frac{dc}{di}$) is required for each pair of measurements.

SECTION A4: COLLECTION AND REDUCTION OF TRAFFIC DATA

HOW TO OBTAIN TRAFFIC DATA

Standard request forms are available from the Planning Survey Division D-10 to obtain estimates of traffic volumes and traffic growth rates. Two traffic classifications are used:

(a) Total traffic (T) in equivalent passenger car units as used in geometric design. In the context of overlay design, total traffic is used (a) to estimate the user delay costs associated with a particular overlay design strategy, and (b) to do lane capacity checks.

(b) Equivalent traffic (E) in 18-k ESAL. The equivalent traffic is used directly in the structural design of overlays.

REDUCTION OF TRAFFIC DATA

Directional and Lane Distribution of Traffic

In order to obtain the traffic in the design lane, traffic data must always be divided between the directions of travel. A 0.5 directional distribution factor (D) does not always apply as demonstrated by actual traffic counts shown in Table A4.1.

For multi-lane roadways, the traffic will be distributed among the lanes. Note that the distribution of total traffic and equivalent traffic will not necessarily be the same. The distribution will also change along the length of a road, depending on geometric factors, climbing lanes or interchange ramps. Suggested design factors for total traffic (L_t) and equivalent traffic (L_e) are given in Table A4.2. As far as possible, these factors should incorporate the change of lane distribution over the geometric life of a facility. The factors should be regarded as maxima and decreases may be justified.

TABLE A4.1 ESTIMATED DIRECTIONAL DISTRIBUTION FACTORS (D) FOR CRCP IN TEXAS (REF.A9)

Highway Section	District	% Failures		% Traffic	
		EB or NB	WB or SB	EB or NB	WB or SB
IH 10	20	32	69	41	59
	24	34	66	42	68
IH 20	10	57	43	53	47
IH 30	1	49	51	49	51
	19	58	42	54	46
IH 35	9	37	63	43	57
IH 45	17	22	78	36	64

TABLE A4.2 DESIGN FACTORS FOR DISTRIBUTION OF TOTAL TRAFFIC AND EQUIVALENT TRAFFIC AMONGST LANES AND SHOULDERS

Total number of traffic lanes	Design distribution factor, L_t or L_e				
	Surfaced slow shoulder	Lane 1*	Lane 2	Lane 3	Surfaced fast shoulder**
(a) Equivalent traffic (E) Factor L_e					
2	1.00	1.00	-	-	-
4	0.95	0.95	0.30	-	0.30
6	0.70	0.70	0.60	0.25	0.25
(b) Traffic (total axles or e.p.u)ø Factor L_t					
2	1.00	1.00	-	-	-
4	0.70	0.70	0.50	-	0.50
6	0.30	0.30	0.50	0.40	0.40

*Lane 1 is the outer or slow lane

**For dual-carriageway roads

øe.p.u = equivalent passenger car unit;
one commercial vehicle = 3 e.p.u.

Projection and Accumulation of Traffic Data

Depending on the design strategy to be evaluated, the traffic in the design lane must be projected to other points in time and accumulated over different time periods. It is therefore necessary to obtain an average daily traffic count at some point in time. Using Table A4.3 (traffic growth factors g_x) and Table A4.4 (cumulative growth factors f_y) the projection and accumulation of traffic data becomes very easy.

Given the average daily traffic at the time of a traffic count, Table A4.3 can be used to project the average daily traffic at any other point in time (in the future or in the past). Multiplying by g_x will project the traffic count to a traffic count at a point in time in the future, while dividing by g_x will shift the traffic could to a point in the past. Table A4.3 is based on the formula:

$$g_x = (1 + 0.01 \times i)^x \qquad \text{(A4.1)}$$

where

g_x = growth factor

i = growth rate

x = time (in years) between determination of traffic count and point in time where the traffic estimate is needed.

Table A4.4 is used to accumulate traffic over a period of time. The average daily traffic at any point in time can be accumulated over a time period (y) in years by multiplying the average daily traffic at the beginning of period y by the cumulative growth factor f_y. The cumulative growth factor f_y given in Table A4.4 is based on the formula:

$$f_y = 365\ (1 + 0.01i) \left[(1 + 0.01i)^y - 1\right] / 0.01i \qquad \text{(A4.2)}$$

TABLE A4.3 TRAFFIC GROWTH FACTOR (g) FOR CALCULATION OF FUTURE OR INITIAL TRAFFIC FROM PRESENT TRAFFIC

TIME BETWEEN DETERMINATION OF AXLE LOAD DATA AND OPENING OF ROAD, x (yrs)	*g FOR TRAFFIC INCREASE, i (% p.a.)								
	2	3	4	5	6	7	8	9	10
1	1.02	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10
2	1.04	1.06	1.08	1.10	1.12	4.14	1.17	1.19	1.21
3	1.06	1.09	1.12	1.16	1.19	1.23	1.26	1.30	1.33
4	1.08	1.13	1.17	1.22	1.26	1.31	1.36	1.41	1.46
5	1.10	1.16	1.22	1.28	1.34	1.40	1.47	1.54	1.61
6	1.13	1.19	1.27	1.34	1.42	1.50	1.59	1.68	1.77
7	1.15	1.23	1.32	1.41	1.50	1.61	1.71	1.83	1.95
8	1.17	1.27	1.37	1.48	1.59	1.72	1.85	1.99	2.14
9	1.20	1.30	1.42	1.55	1.69	1.84	2.00	2.17	2.36
10	1.22	1.34	1.48	1.63	1.79	1.97	2.16	2.37	2.59
11	1.24	1.38	1.54	1.71	1.90	2.10	2.33	2.58	2.85
12	1.27	1.43	1.60	1.80	2.01	2.25	2.52	2.81	3.14
13	1.29	1.47	1.67	1.89	2.13	2.41	2.72	3.07	3.45
14	1.32	1.51	1.73	1.98	2.26	2.58	2.94	3.34	3.80
15	1.35	1.56	1.80	2.08	2.40	2.76	3.17	3.64	4.18
16	1.37	1.60	1.87	2.18	2.54	2.95	3.34	3.97	4.59
17	1.40	1.65	1.95	2.29	2.69	3.16	3.70	4.33	5.05
18	1.43	1.70	2.03	2.41	2.85	3.38	4.00	4.72	5.56
19	1.46	1.75	2.11	2.53	3.03	3.62	4.32	5.14	6.12
20	1.49	1.81	2.19	2.65	3.21	3.87	4.66	5.60	6.73
21	1.52	1.86	2.28	2.79	3.40	4.14	5.03	6.11	7.40
22	1.55	1.92	2.37	2.93	3.60	4.43	5.44	6.66	8.14
23	1.58	1.97	2.46	3.07	3.82	4.74	5.87	7.26	8.95
24	1.61	2.03	2.56	3.23	4.05	5.07	6.34	7.91	9.85
25	1.64	2.09	2.67	3.39	4.29	5.43	6.85	8.62	10.83
26	1.67	2.16	2.77	3.56	4.55	5.81	7.40	9.40	11.92
27	1.71	2.22	2.88	3.73	4.82	6.21	7.99	10.25	13.11
28	1.74	2.29	3.00	3.92	5.11	6.65	8.63	11.17	14.42
29	1.78	2.36	3.12	4.12	5.42	7.11	9.32	12.17	15.86
30	1.81	2.43	3.24	4.32	5.74	7.61	10.06	13.27	17.45

$^{*}g = (1 + 0.01i)^{x}$

TABLE A4.4 TRAFFIC GROWTH FACTOR (f) FOR CALCULATION OF CUMULATIVE TRAFFIC OVER PREDICTION PERIOD FROM INITIAL (DAILY) TRAFFIC

PREDICTION PERIOD, y (YRS)	COMPOUND GROWTH RATE, i (% p.a.)											
	1	2	3	4	5	6	7	8	9	10	11	12
4	1 500	1 530	1 570	1 610	1 650	1 690	1 730	1 780	1 820	1 860	1 910	1 950
5	1 880	1 940	2 000	2 060	2 120	2 180	2 250	2 310	2 380	2 380	2 520	2 600
6	2 270	2 350	2 430	2 520	2 610	2 700	2 790	2 890	2 990	3 100	3 210	3 320
7	2 660	2 770	2 880	3 000	3 120	3 250	3 380	3 520	3 660	3 810	3 960	4 120
8	3 050	3 200	3 340	3 500	3 660	3 830	4 010	4 190	4 390	4 590	4 800	5 030
9	3 450	3 630	3 820	4 020	4 230	4 450	4 680	4 920	5 180	5 450	5 740	6 040
10	3 860	4 080	4 310	4 560	4 820	5 100	5 400	5 710	6 040	6 400	6 770	7 170
11	4 260	4 530	4 820	5 120	5 440	5 790	6 160	6 560	6 990	7 440	7 930	8 440
12	4 680	4 990	5 340	5 700	6 100	6 530	6 990	7 480	8 010	8 590	9 200	9 870
13	5 090	5 470	5 870	6 310	6 790	7 310	7 870	8 470	9 130	9 850	10 600	11 500
14	5 510	5 950	6 420	6 940	7 510	8 130	8 810	9 550	10 400	11 200	12 200	13 200
15	5 930	6 440	6 990	7 600	8 270	9 010	9 810	10 700	11 700	12 800	13 900	15 200
16	6 360	6 940	7 580	8 280	9 070	9 930	10 900	12 000	13 100	14 400	15 900	17 500
17	6 790	7 450	8 180	9 000	9 900	10 900	12 000	13 300	14 700	16 300	18 000	20 000
18	7 230	7 970	8 800	9 730	10 800	12 000	13 300	14 800	16 400	18 300	20 400	22 800
19	7 670	8 500	9 440	10 500	11 700	13 100	14 600	16 300	18 300	20 500	23 100	25 900
20	8 120	9 050	10 100	11 300	12 700	14 200	16 000	18 000	20 400	23 000	26 000	29 500
25	10 400	11 900	13 700	15 800	18 300	21 200	24 700	28 800	33 700	39 500	46 400	54 500
30	12 800	15 100	17 900	21 300	25 500	30 600	36 900	44 700	54 200	66 000	80 600	98 700
35	15 400	18 600	22 700	28 000	34 600	43 100	54 000	67 900	85 800	109 000	138 000	176 000
40	18 000	22 500	28 300	36 100	46 300	59 900	78 000	102 000	134 000	178 000	236 000	314 000

Based on $f = 365.\ (1 + 0.01.i).\ [(1 + 0.01.i)^{y} - 1]/(0.01.i)$

where

f_y = cumulative growth factor

i = yearly growth rate

y = accumulation period in years.

<u>Example to Demonstrate the Use of Tables A4.3 and A4.4</u>

Given: The present average daily equivalent traffic (E) in the design lane = 1,000 18k - ESAL. The equivalent traffic growth rate = 3 percent. A road is to be overlaid 2 years from now and the overlay is to be designed for 20 years.

Required: Determine the cumulative design equivalent traffic N_{18}

N_{18} = $1000(g_2)(f_{20})$ ... g_2 and f_{20} from Table A4.3 and A4.4 respectively

= 1000(1.06)(10100)

= 10,706,000 18-K ESAL

LANE CAPACITY CHECKS

In order to check the geometric capacity of the road, the total daily traffic towards the end of the structural design period could be calculated using the formula:

$$N = (\text{initial total daily traffic}) \quad g_x \qquad \text{(A4.3)}$$

with g_x as previously defined (Table A4.3).

When projecting traffic over the structural design period, the designer should take into account the possibility of capacity conditions being reached, resulting in no further growth in traffic for that particular lane.

SECTION A5: BASIC DESIGN CRITERIA

Design criteria set the framework, in the form of constraints, within which the rehabilitation must be designed. These include:

Physical or Geometric Constraints

These include width and height constraints such as bridge openings, as well as capacity constraints.

Time Constraints

Construction time limitations, times between overlays and the analysis period fall under this heading.

Budget Constraints

The budget constraints can for example have significant influence on the number of viable design strategies.

Traffic Constraints

Traffic variables are important design criteria and apart from being direct design inputs, they must also be considered in the planning and scheduling of the construction work.

Material and Construction Constraints

Traffic variables are important design criteria and apart from being direct design inputs, they must also be considered in the planning of the construction process.

Material and Construction Constraints

The materials economically available as well as the available construction experience can potentially limit the number of viable alternative design strategies.

It is important for the designer to take timely note of these constraints to avoid embarrassment and additional work. When a constraint is unrealistic, it is the responsibility of the designer to go back to the client and explain the situation.

SECTION A6: SELECTION OF DESIGN SECTIONS

DESIGN SECTIONS

A highway can be divided into different design sections, on the basis of e differences in deflection test data. Each design section then becomes a separate design problem.

SOURCES OF INFORMATION

Two basic sources of information are generally used for the selection of design sections. These are:

(1) surface deflections, and
(2) condition survey information.

STEPS TO FOLLOW

Figure A6.1 presents a flow diagram of the steps involved in the selection of design sections. Two deflection parameters are recommended for use. These are the midspan Dynaflect sensors (W_5) deflection and the deflection slope ($W_1 - W_5$) i.e., the Dynaflect sensor 1 deflection minus the sensor 5 deflection. The following steps are recommended:

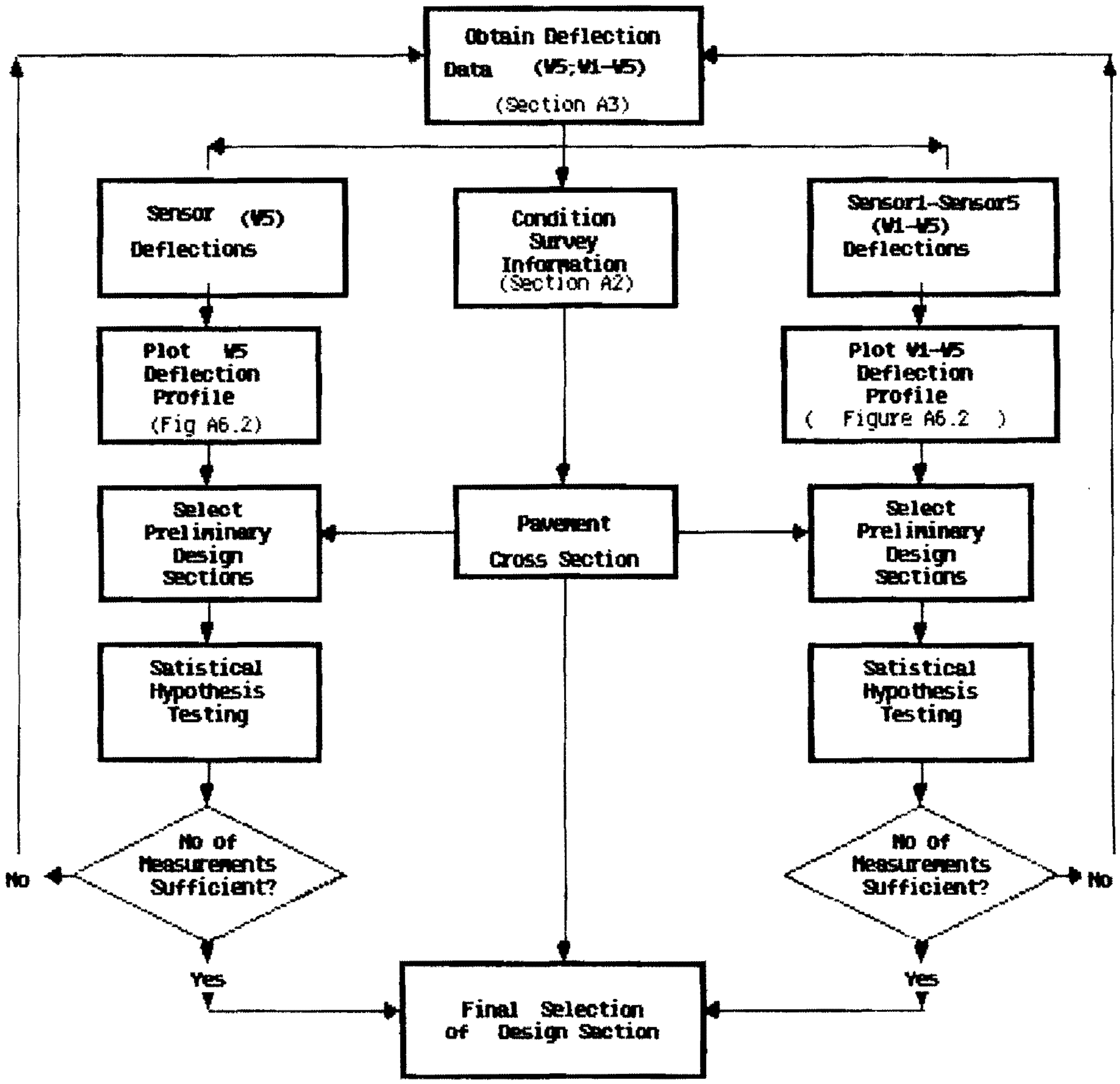


Fig A6.1. Steps involved in the selection of design sections.

Step 1: Deflection Profiles

The deflection parameters (W_5 and $W_1 - W_5$) are plotted in the form of profiles throughout the length of the roadway as shown in Fig A6.2. The plots can be made manually or by using the computer program PLOT2 (Ref A1).

Step 2: Preliminary Design Sections

The deflection profiles are divided into areas which have similar deflection parameters based on stratified variation in deflection data. The sensor 5 deflection is used to select sections with different subgrade stiffness and the basin slopes are used to select sections with different effective surface stiffness. Sections are selected subjectively, based on a plotted profile of these deflection parameters, as indicated in Fig A6.2. In order to keep the number of sections to a minimum, limits of section selected from the two deflection parameters should be made to coincide whenever possible. Areas which have significantly different cross-sections should be assigned different sections of deflection profile.

Step 3: Statistical Hypothesis Testing

Adjacent design sections which have the same cross-section should be tested to determine whether they are significantly different or whether they are from the same population. The tests should be done for each set of preliminary design sections (as determined from the W_5 and $W_1 - W_5$ deflection profile) separately. The student t-test for equal means is recommended for this purpose. The student t-test, however, assumes that the sections being tested have similar variances. If this is not true or if there is some concern about this in this regard, a statistical test designed for testing differences between sample variances should be used.

Step 4: Check Sample Size

Sufficient deflection measurements must be available to allow the designer to make fairly accurate inferences about the section's overall

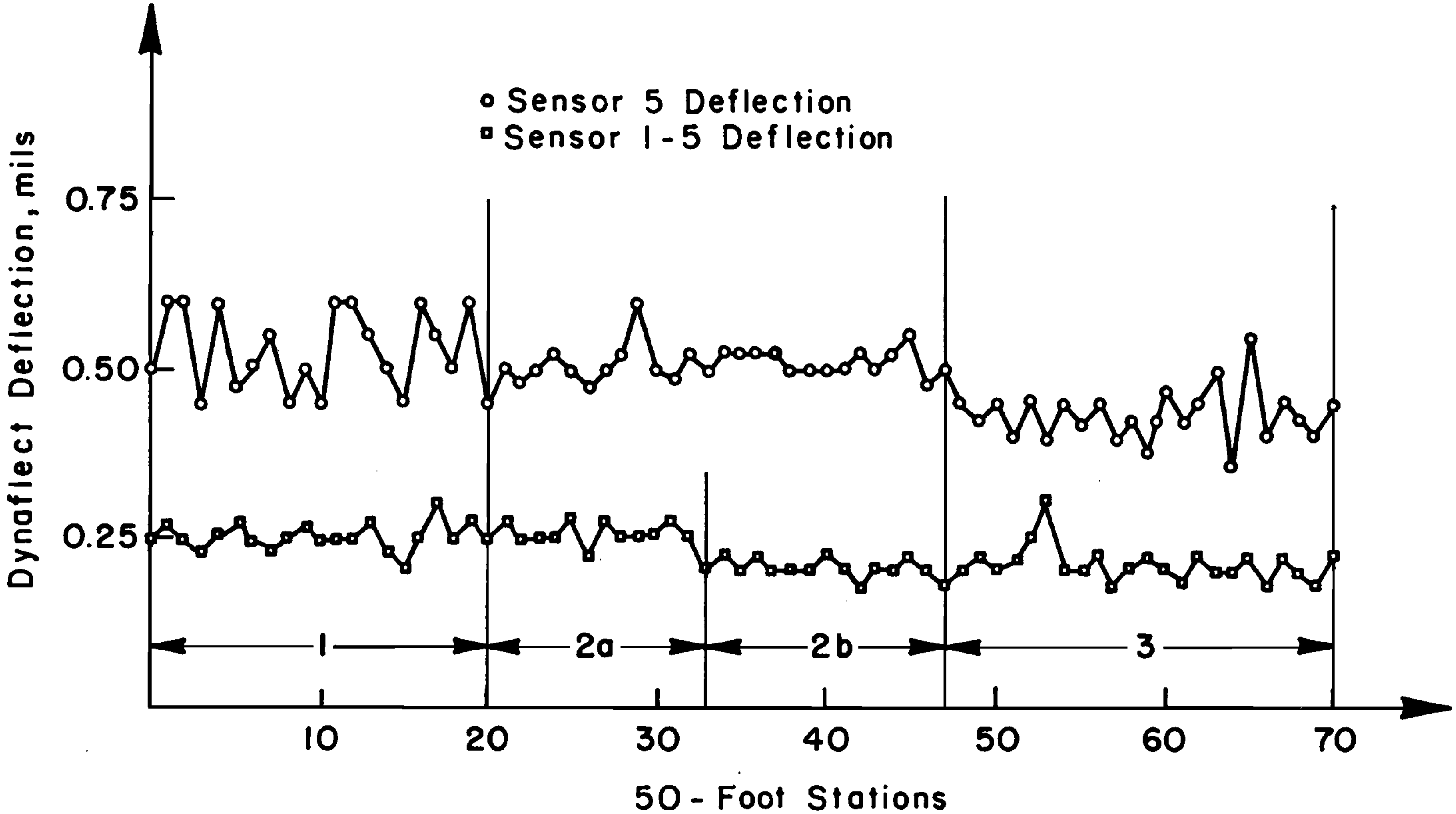


Fig A6.2. Selection of design sections using Dynaflect sensor 1 and sensor 1 minus sensor 5 deflections. (Ref A7).

behavior from the sample of deflections. The procedure to check sample size is outlined in Section A3.

Step 5: Final Selection of Design Sections

Finally, contiguous design sections are selected through the combination of relevant condition survey data and the preliminary sections selected from the W_5 and $W_1 - W_5$ deflection data. Implementation of the Texas Rigid Pavement Design Procedure has indicated that the minimum section length should be approximately 1,000 feet to ensure a practical construction unit length.

SECTION A7: STRUCTURE AND MATERIAL PROPERTIES OF EXISTING PAVEMENT

INFORMATION REQUIRED

The following inputs are required for rehabilitation design:

(1) layer thicknesses and material type; and
(2) elastic properties (E-values) of each layer in the structure.

Layer Thicknesses and Material Type

The layer thicknesses and basic material types can be obtained from construction records and should preferably be available at the start of the project analysis. This information can be verified by direct inspection and measurement, if materials samples are taken from the section for laboratory testing.

Elastic Properties (E-values) of the Existing Pavement Layers

Three sources of information are available to estimate the E-values of the existing pavement. These are:

(1) laboratory test data of samples taken from the design section,

(2) typical E-values of materials in the region, and

(3) deflection data from Section A3.

Both graphical and computerized methods are available. The graphical methods are based on a three-layer structure and described below. For more complex structures as well as for more accurate estimates of layer moduli for 3-layer structures, the computerized methods are recommended and reference A3 may be consulted. Figure A7.1 shows how these sources of data can be combined manually to arrive at a set of E-values to use in the overlay analysis. Not all three sources may be available and the design engineer may have to rely on the available sources for his estimates.

Laboratory Tests. Table A7.1 summarizes standard laboratory tests which are recommended to obtain the E-values of the different material types in the structure.

Typical E-values of Similar Materials in the Region. Another very useful source of information is the E-values obtained from previous analyses in the region. It is recommended that such a table be developed in each region. Table A7.2 summarizes typical E-values of materials generally used for road construction in Texas.

E-value Estimates from Deflection Data. Two deflection statistics are recommended for use in estimating the E-values of the layers in the existing pavement structure. These are:

(1) Dynaflect sensor 5 deflection (W_5), and

(2) Dynaflect deflection slope (W_1 - W_5). The whole deflection profile should be used for more accurate analysis when elastic layer programs are used.

The following procedure is recommended:

(1) Estimation of Subgrade Modulus (E_3). The steps to be followed, are:

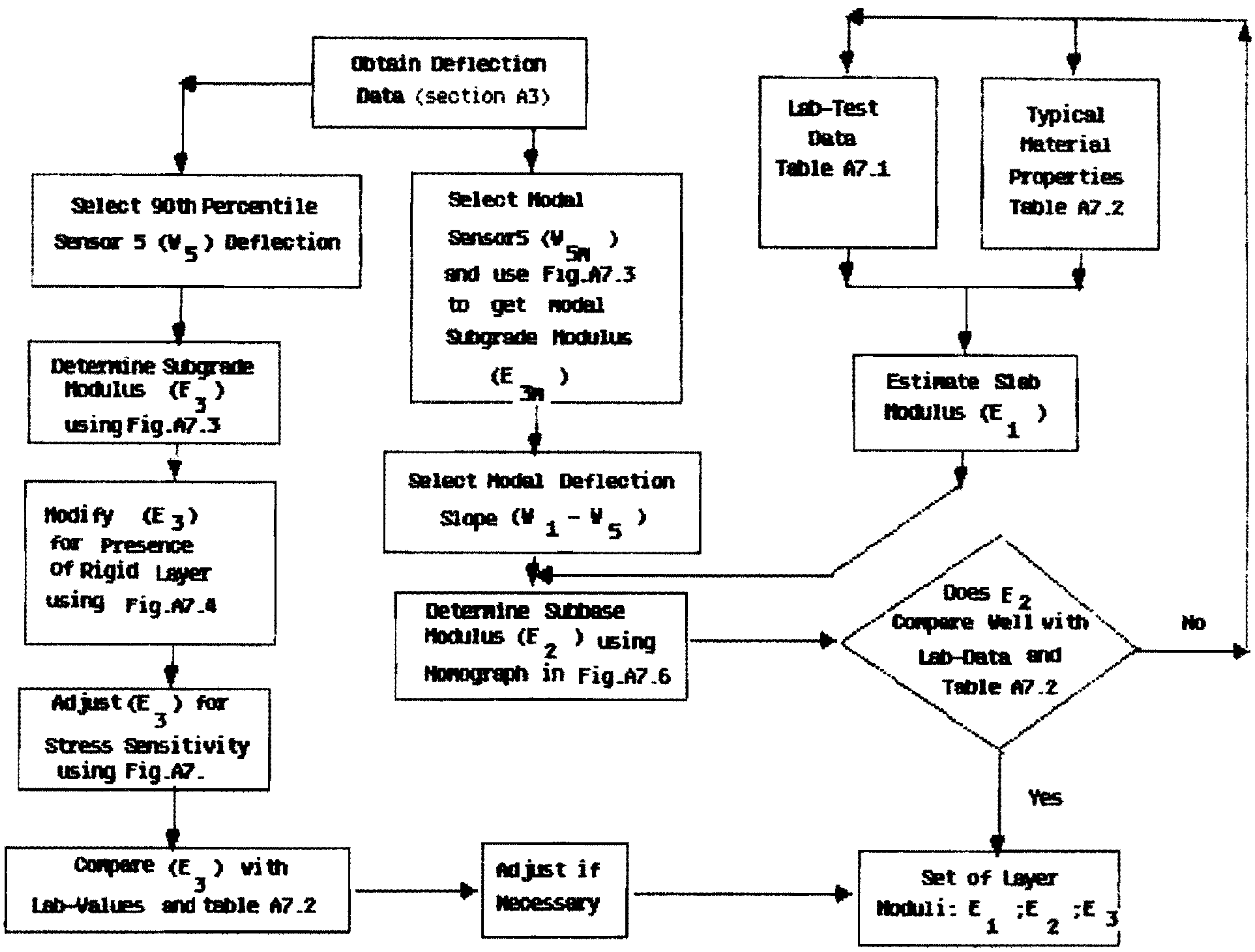


Fig A7.1. Procedure for characterizing the elastic material properties of the existing pavement.

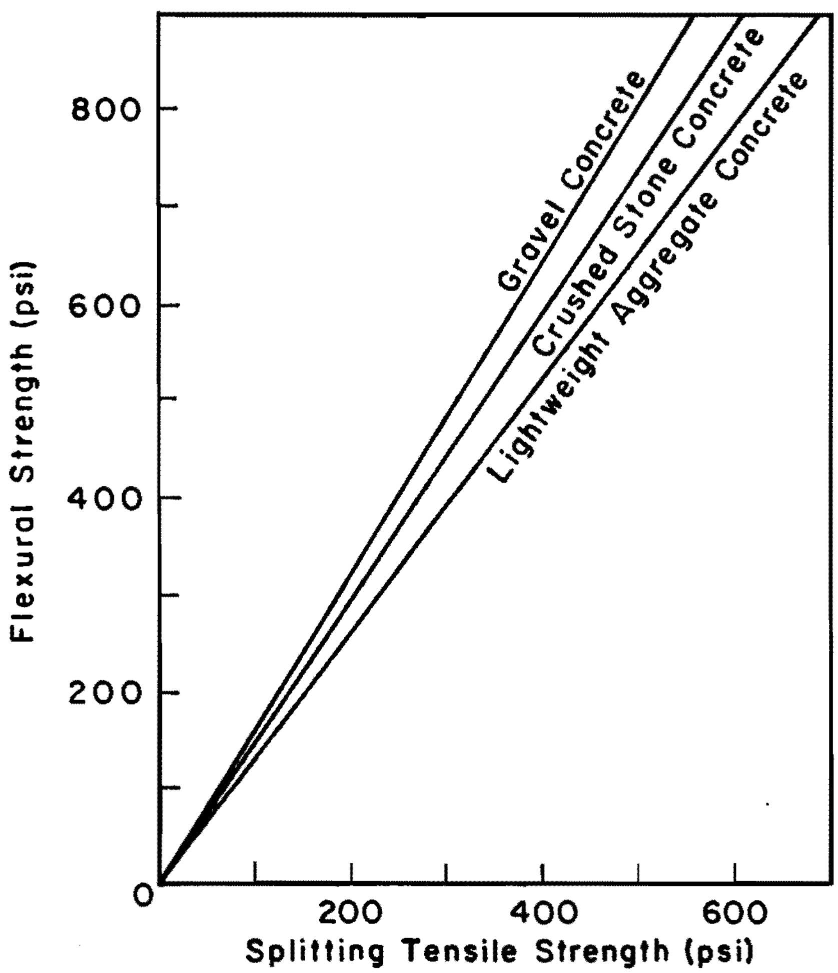


Fig A7.2. Relationship between flexural strength and splitting tensile strength for concrete made with three different aggregates (Ref A1).

TABLE A7.1 STANDARD LABORATORY TESTS TO DETERMINE MATERIAL PROPERTIES REQUIRED FOR OVERLAY DESIGN

	MATERIAL TYPE			
Material property	PCC Slab	Subbase		Subgrade
		Stabilized	Unstabilized	
E-value	Dynamic indirect tensile test Ref. (A 10)	Dynamic indirect tensile test Ref. (A 10)	Resilient modulus test Ref. (A 10)	Resilient modulus test Ref. (A 10)
Flexural strength	Static indirect tensile test* Ref. (A10)	-	-	-

*Adjust indirect tensile strength to flexural strength Strength using Figure A7.2

TABLE A7.2 TYPICAL E-VALUES OF PAVEMENT MATERIALS

LAYER	MATERIAL	CONDITION			
		Uncracked		Cracked	
		E-value (PSI)	Flex strength	E-value (PSI)	Flex strength
PCC SLAB	Coarse aggregate type:				
	- River gravel	$5x10^6$-$7x10^6$	550-850	800 000	
	- Lime stone	$3.5x10^6$-$5x10^6$	400-550	400 000	
SUBBASE		Stabilized		Unstabilized	
		E-value (PSI)		E-value (PSI)	
	Granular	$0.5x10^5$-$20x10^5$		$30x10^3$-$60x10^4$	
SUBGRADE		Resilient modulus (Mr) PSI*			
	Cohesive clay type	$3 \times 10^3 - 4 \times 10^3$			
	Fine grained sandy soil	$20 \times 10^3 - 30 \times 10^3$			
	Lime treated subgrade layer	$5 \times 10^4 - 30 \times 10^4$			

*Adjustments for stress sensitivity necessary (Figure A7.5)

(a) Select the design W_5 deflection from deflection data of section. The 90th percentile value of the midspan sensor 5 deflections is recommended for use as design W_5.

(b) Obtain the subgrade modulus E_3 from Fig A7.3 using the design W_5 deflection as an input.

(c) Adjust the E_3 value obtained in (b) to account for the presence of a rigid layer at shallow depth below the subgrade using Fig A7.4 and stress sensitivity using Fig A7.5. If laboratory data is available, Fig A7.5 can be adjusted for more accurate estimates. Again elastic layer computer programs can be used for more accurate modelling of a rigid layer.

(d) Compare the E_3 value obtained in (c) with subgrade moduli of similar materials in the region (Table A7.2) and laboratory data if available and adjust if necessary. Adjustments of this kind may for example be justified to account for seasonal variations in subgrade moduli.

(2) Estimation of PCC slab Modulus (E_1) and Subbase Modulus (E_2). The steps to be followed, are:

(a) Select the modal sensor 5 deflection W_{5m} from deflection data of the design section. The modal deflection is the deflection that occurs most frequently in the section.

(b) Use Fig A7.3 to obtain the modal E_3 value (E_{3m}).

(c) Select the design deflection slope (W_1 - W_5) from the deflection data of the section. The modal value of the midspan deflection is recommended for use.

(d) Obtain a first estimate of the slab modulus (E_1) from slab moduli of similar materials in the region (Table A7.2) and laboratory data when available.

(e) Use the E_{3m}, slope (W_1 - W_5), and the E_1 estimate obtained in (b), (c), and (d) and the nomograph in Fig A7.6 to obtain a first estimate of the subbase modulus (E_2).

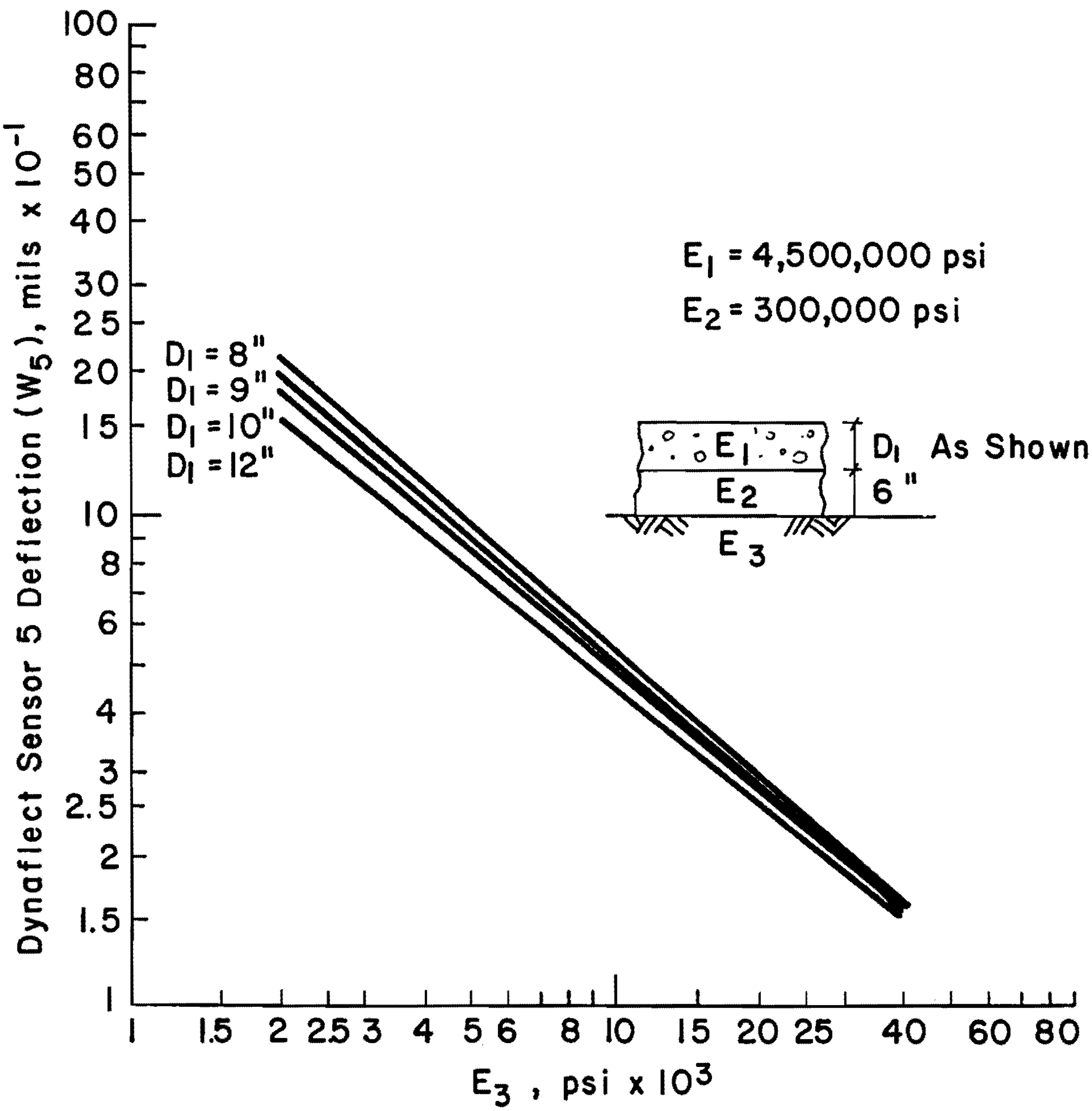


Fig A7.3. Dynaflect sensor 5 - subgrade modulus relationship for different rigid pavement thickness (Ref A2).

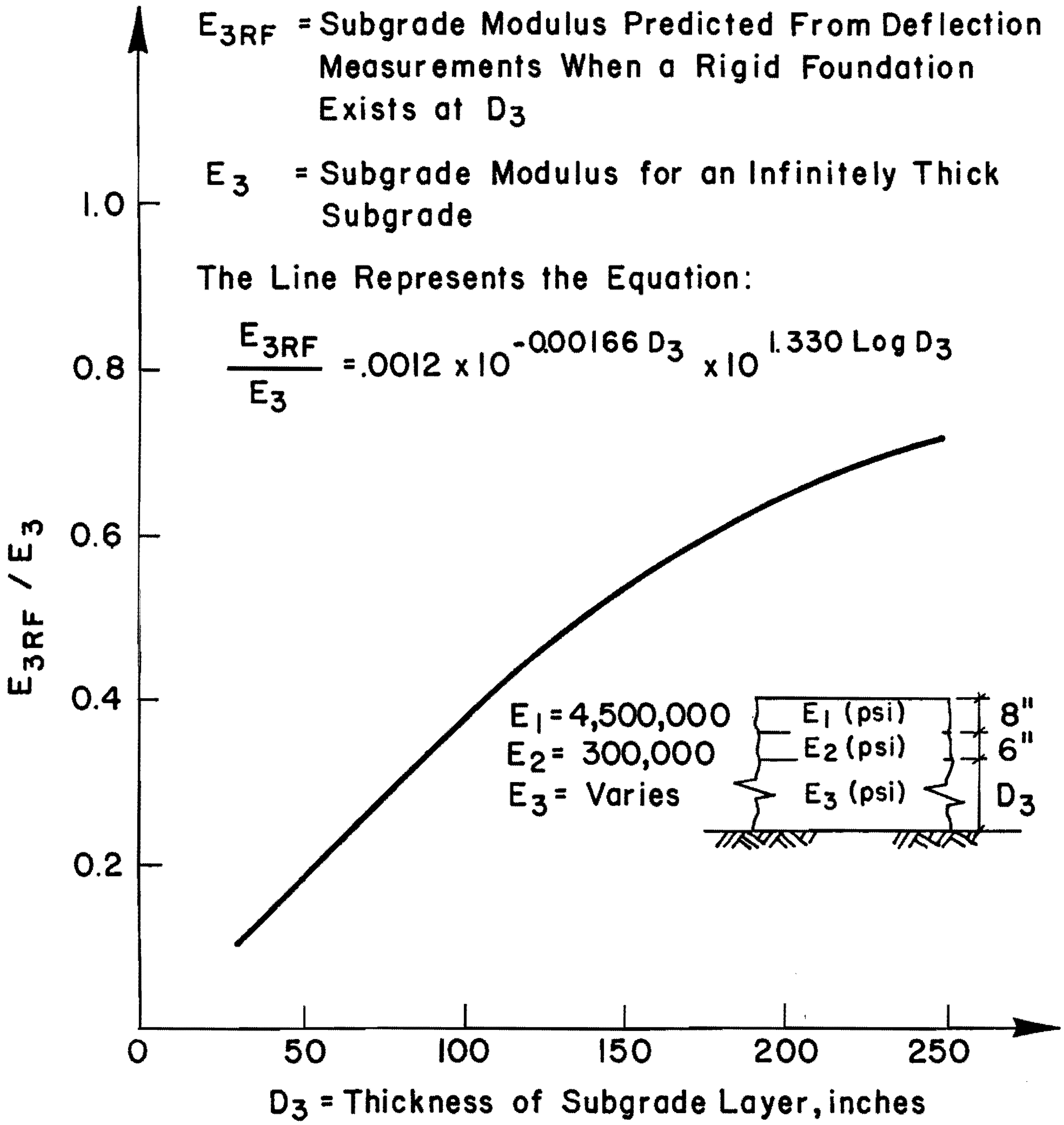


Fig A7.4. The reduction in subgrade modulus predicted using deflection measurements when the subgrade is supported by a rigid foundation at depth D_3 (Ref A2).

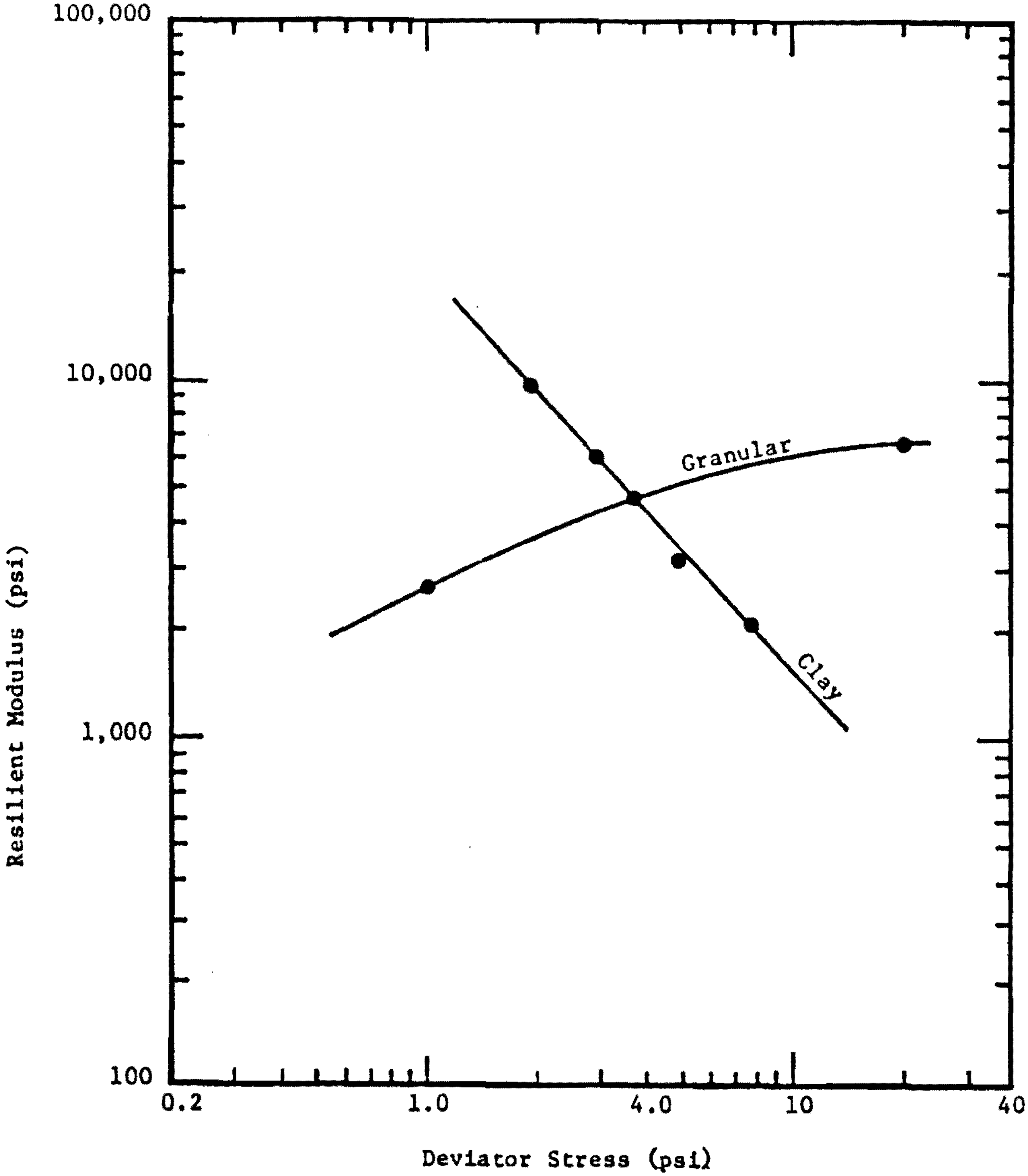


Fig A7.5. **Relationship between resilient modulus and stress for typical clay and granular soils (Ref** A1).

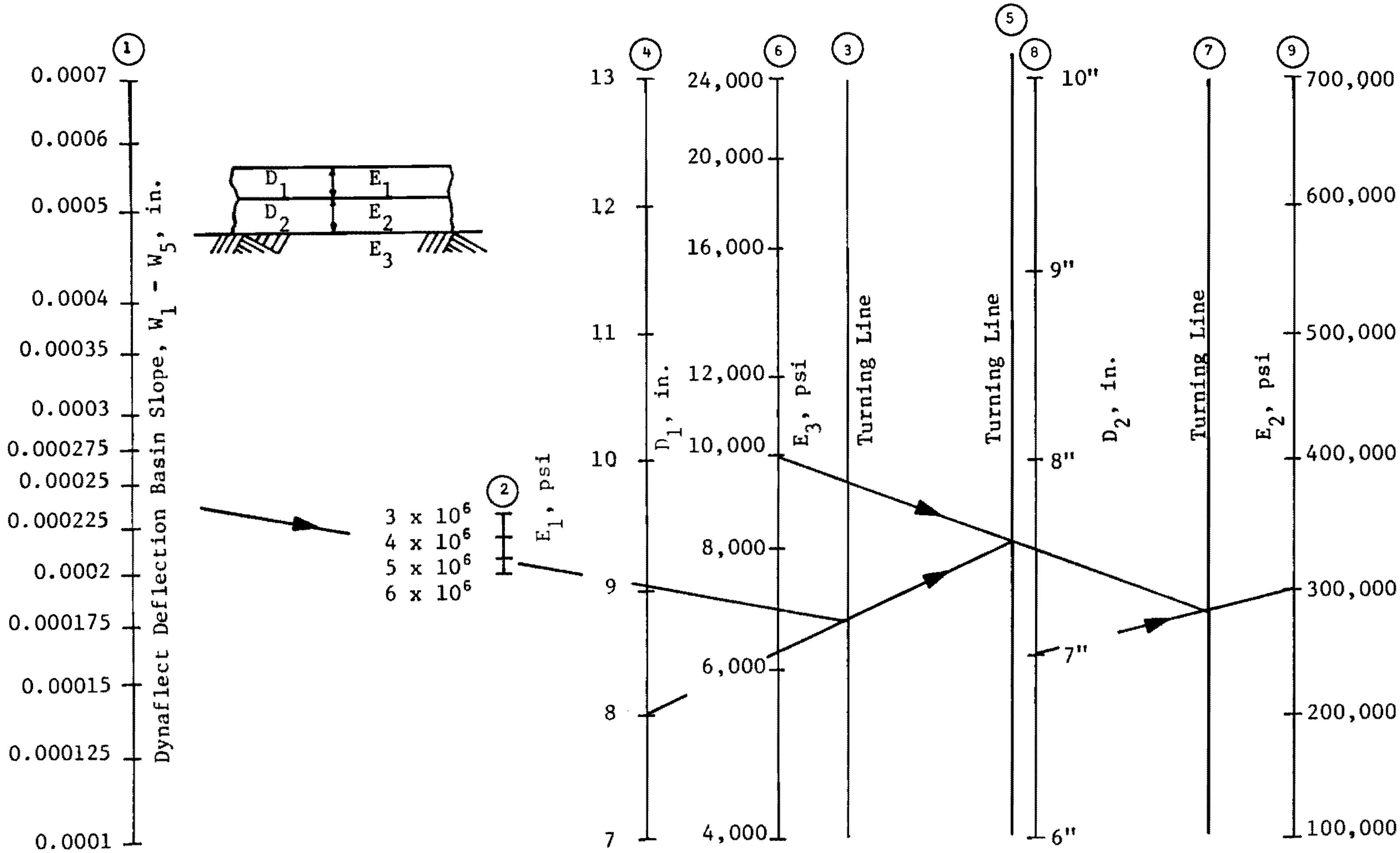


Fig A7.6. Nomograph for estimating subbase modulus of elasticity (E_2) for rigid pavements from Dynaflect deflections (Ref 2).

(f) Compare the E_2 value with subbase moduli of similar materials in the region (Table A7.2) and laboratory data when available. If not satisfied, repeat steps (d) through (f).

PCC FLEXURAL STRENGTH

Two basic sources of information can be consulted to obtain the PCC flexural strength. These are laboratory test data on samples taken from the existing PCC slab and flexural strength values of similar materials in the region (refer to Tables A7.1 and A7.2). The third point flexural strength is used in the design procedure. When 7 or 28 day PCC strength data are used to estimate the flexural strength, these values must be adjusted to 90 day flexural strengths.

Since it is not always feasible to obtain beam samples from the existing PCC slab, 4 inch cores are normally tested and the midspan relationship between indirect tensile strength and flexural strength shown in Fig A7.2 may be used to predict the flexural strength.

PRODUCT OF THIS SECTION

This section should provide the following information:

(1) Thicknesses for each layer.
(2) E-values for each layer.
(3) Concrete flexural strength.

SECTION A8: ESTIMATION OF REMAINING LIFE AND FINAL CALIBRATION OF EXISTING PAVEMENT PROPERTIES FOR USE IN MECHANISTIC MODEL

The estimation of the remaining life of the existing pavement is complex but very important, and a lot of engineering judgement is needed in defining this value. The mechanistic design model used in this manual, defines the remaining life of an existing pavement as remaining fatigue life using the following equations:

$$RL = \left(1 - \frac{n_{18}}{N_{18}}\right) \times 100 \qquad \text{(A8.1)}$$

where

RL = percent remaining life

n_{18} = accumulated past traffic in 18k ESALS

N_{18} = original (or design) fatigue life of existing pavement in 18k ESALS.

The most important information available for the estimation of remaining is the present condition of the pavement. It is thus clear that the mechanistic fatigue model and the structural performance history of the pavement must be merged in this section. The specific objectives of this section, thus, are twofold:

(1) To estimate the remaining life of the existing pavement.

(2) To "calibrate" the mechanistic fatigue model by adjusting the inputs of the model (i.e., the properties of the existing pavement) to make the predictions of the mechanistic model compatible with the performance history of the pavement.

RECOMMENDED PROCEDURE

Two estimates of remaining life are made:

(1) RL_1 based on the present condition of the pavement, pavement age and accumulated past equivalent 18k ESAL.

(2) RL_2 based on the mechanistic fatigue model using the material properties obtained in Section A6 as inputs.

The two estimates are then compared and adjustments made to converge to representative values. Though this is an approximate method but should avoid major discrepancies between predictions of the mechanistic model and actual structural performance of the pavement. Figure A8.1 shows the steps involved in this procedure.

Remaining Life 1 (RL_1)

Steps to be followed are:

(1) Calculate distress index (Z value) of the existing pavement using condition survey data (Section A2).

(2) Obtain the age of the existing pavement in years from past records.

(3) Calculate the accumulated past traffic of the design lane (n_{18}) in 18k ESALs (Section A4).

(4) Enter the nomograph (Fig A8.2) with the information on the z-value and past traffic to obtain RL_1. The estimate obtained from the nomograph may be refined through interpolation and engineering judgement based on local experience of the relationship between present condition and expected future structural performance of existing pavements. An existing pavement is considered to have reached the end of its structural life (i.e., RL = 0) when the rate of defect development (in defects per mile per year) is in the order of 3 to 4 per year.

Remaining Life 2 (RL_2)

Steps to be followed are:

(1) Obtain the accumulated past equivalent traffic (n_{18} in 18k ESAL) from Section A4, and the set of material properties and layer thicknesses of the existing pavement estimated in Section A7.

(2) Compare the PCC slab modulus (E_1) with E_1-values of similar materials in the region.

(3) Using the above comparison, judgement should be exercised to decide whether the slab appear to be fatigued. This will be true if the

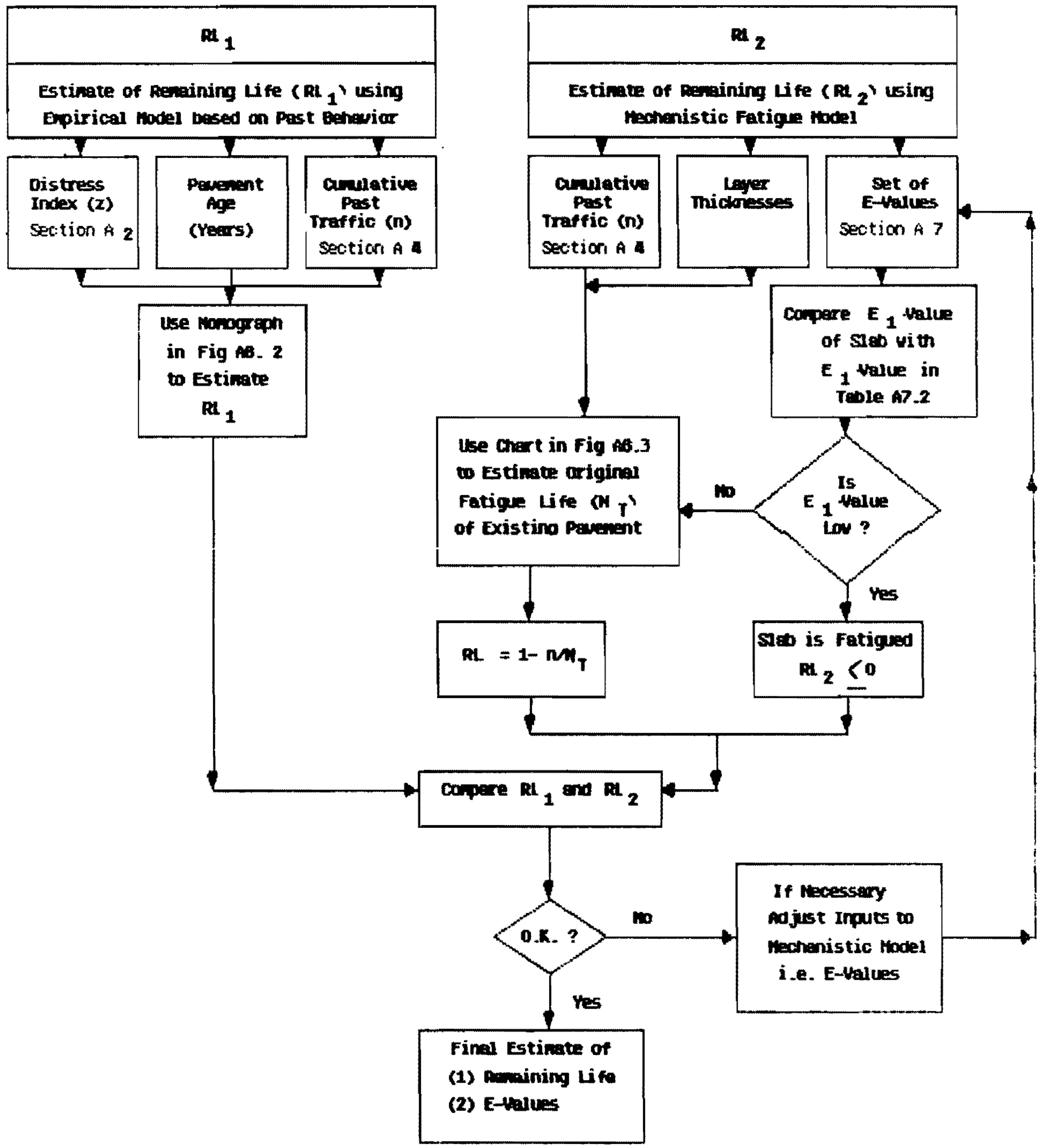


Fig A8.1. Steps involved in the estimation of the remining life of the existing pavement.

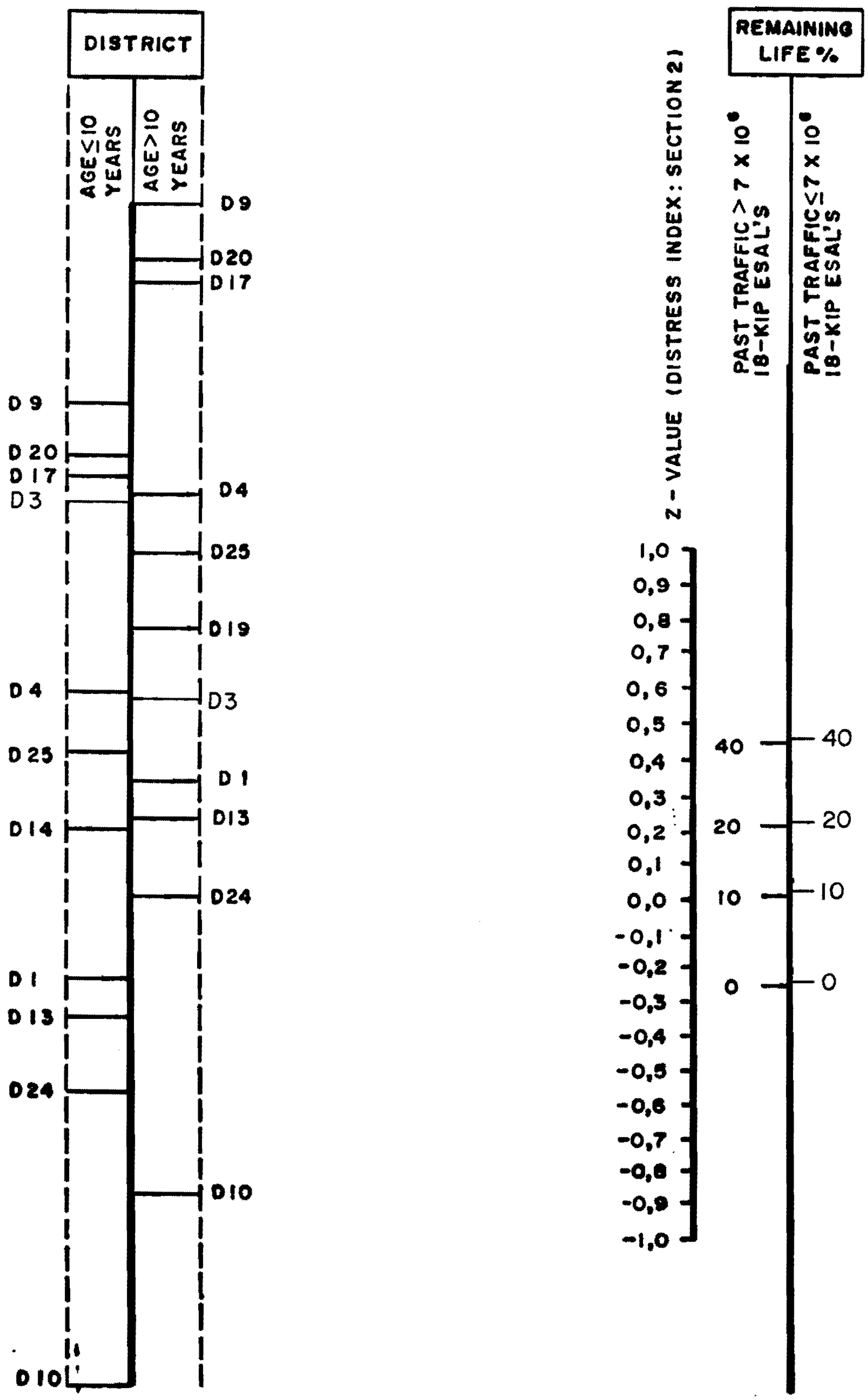


Fig A8.2. Nomograph to determine life as a function of district, pavement age, distress index and accumulated past equivalent traffic.

slab modulus (E_1) obtained in Section A7 is substantially lower than that of similar materials in the region.

(4) If the PCC slab appear to be fatigued, no further mechanistic analysis is possible at this point and the estimate of remaining life 2 (RL_2) is simply set equal to zero.

(5) If the slab modulus (E_1) is relatively high compared to the moduli of similar materials in the region the original structural design, the remaining life of the existing pavement structure must be calculated. In order to do this, the tensile stress in the PCC slab must be calculated using an elastic layer program such as ELSYM5 (Ref A4). A set of regression equations (Ref A5), ideal for use in a programmable hand calculator, is also available for this purpose (Refer to Appendix C). The stress obtained is then adjusted using to the critical stress (σ_c) by multiplying with the appropriate stress factor selected from Table A8.1.

The original design fatigue life (N_{18}) is then calculated using Eq A.2

$$N_{18} = 46000 \left(\frac{f}{\sigma_c}\right)^{3.0} \qquad \text{(A8.2)}$$

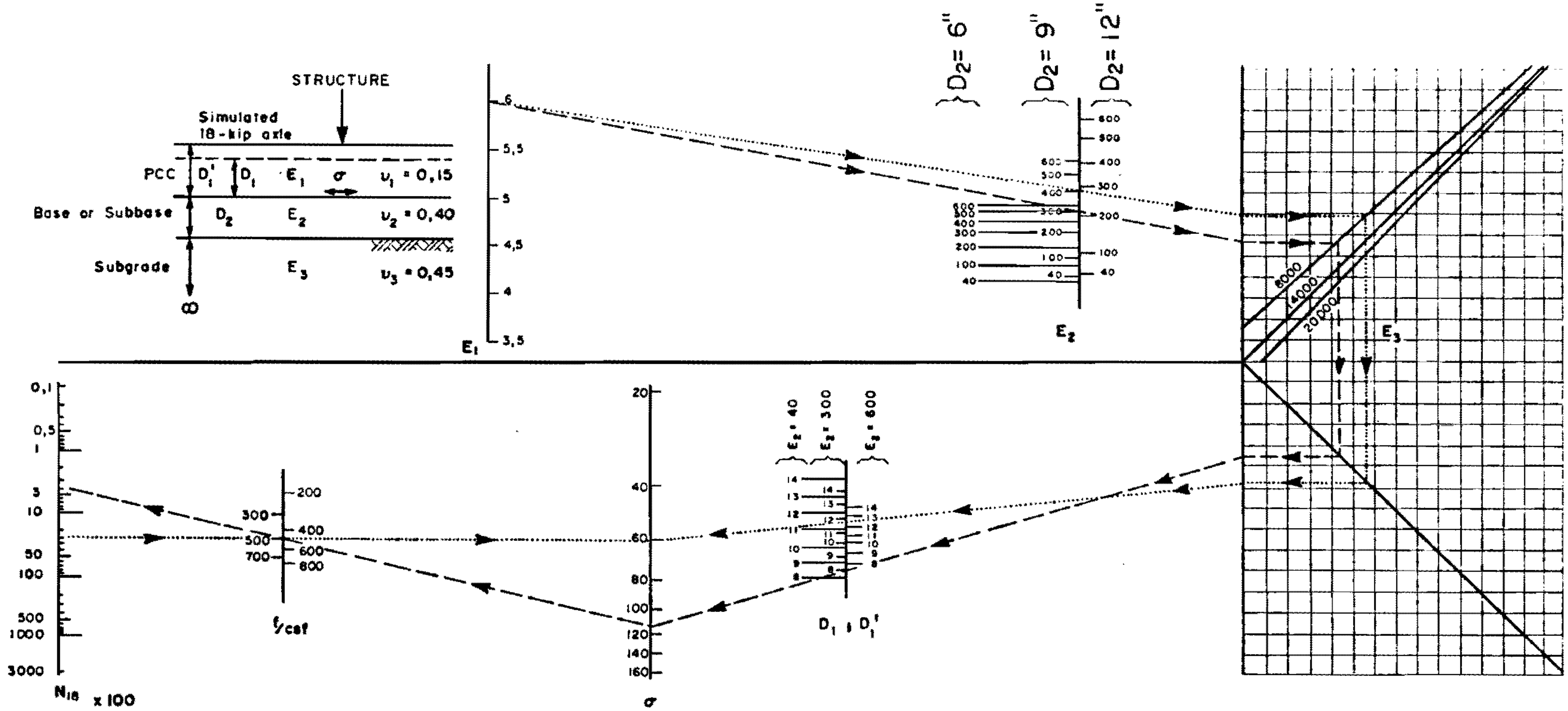
where

N_{18} = original design fatigue life in 18k ESAL

f = concrete flexural strength

σ_c = critical stress

For a three-layer structure the original structural design life of the existing pavement can be obtained directly from the chart in Fig A8.3.



WHERE :	APPLICATION	
N_{18} = NUMBER OF 18-KIP ESAL APPLICATIONS TO TERMINAL DISTRESS CONDITION (10^6 18-KIP ESAL)	1 CALCULATE ORIGINAL DESIGN LIFE OF EXISTING PAVEMENT (ASSUME RL = 100%)	2. DETERMINE THE REQUIRED THICKNESS OF A THIN BONDED PCC OVERLAY (R.L. OF EXISTING PAVEMENT TO BE SPECIFIED)
R.L. = PERCENT REMAINING LIFE OF EXISTING PAVEMENT f = FLEXURAL STRENGTH OF PCC csf = CRITICAL STRESS FACTOR E_1 = PCC - MODULUS (10^6 psi) E_2 = SUBBASE - MODULUS (10^3 psi) E_3 SUBGRADE - MODULUS (psi) D_i = LAYER THICKNESSES (INCHES)	EXAMPLE (a) GIVEN : $E_1 = 6 \times 10^6$ psi ; $E_2 = 300 \times 10^3$ psi ; $E_3 = 8000$ psi $D_1 = 8''$; $D_2 = 9''$; $f = 600$ psi ; CSF = 1,2 (b) REQUIRED: ORIGINAL DESIGN LIFE (c) ANSWER: $N_{18} = 3,9 \times 10^6$ 18 - KIP ESAL (REFER TO BROKEN LINE --------)	EXAMPLE (a) GIVEN : $E_1 = 6 \times 10^6$ psi ; $E_2 = 400 \times 10^3$ psi ; $E_3 = 8000$ psi ; $D_1 = 8''$; $D_2 = 9''$; f = 600 psi ; CSF = 1,2 ; RL = 40% (b) REQUIRED: THICKNESS OF THIN BONDED PCC - OVERLAY TO CARRY 10×10^6 18-KIP ESAL (c) ANSWER : $\frac{N}{RL} \times 100 = \frac{10 \times 10^6}{40} \times 100 = 25 \times 10^6$ $D_1' = 12''$ (REFER TO DOTTED LINE ....) $\therefore$ REQUIRED OVERLAY THICKNESS = $D_1' - D_1 = 4''$

Fig A8.3. Chart to (a) calculate original design life of existing pavement and (b) determine the required thickness of a thin bonded PCC overlay.

TABLE A8.1 EXISTING PAVEMENT CRITICAL STRESS FACTORS (REF.A5)

Existing Pavement Type	Existing PCC shoulders	Range of critical stress factor
CRCP	No	1.20 - 1.25
	Yes	1.05 - 1.10
JCP (with load transfer)	No	1.25 - 1.30
	Yes	1.10 - 1.20
JCP (without load transfer)	No	1.50 - 1.60
	Yes	1.40 - 1.50

(6) The estimate of RL_2 is then calculated using Eq A8.1:

$$RL_2 = (1 - \frac{n_{18}}{N_{18}}) \times 100 \tag{A8.3}$$

<u>Comparison of RL_1 and RL_2</u>. The two estimates of remaining life, RL_1 and RL_2, are compared. If gross discrepancies occur, adjustments will be necessary. The nature of the adjustment will depend on the reliability of the data used to obtain RL_1 or RL_2. Engineering judgement should be exercised and the following points may be kept in mind:

(a) More than one slab/subbase modulus ratio (E_1/E_2) can satisfy the deflection basin (W_1 - W_5) used in Section A7. If uncertainty exists around the ratio estimated in Section A6, this section will provide a further piece of information.

(b) Seasonal variations occur that will affect both material properties an surface deflections. The set of E-values used in the mechanistic model should attempt to reflect the effect of these variations as well as variability of pavement materials in the long run.

(c) Higher critical stress factors may apply if voids are present under the existing slab. Severe pumping at the discontinuities may indicate the presence of voids.

FINAL PRODUCT OF THIS SECTION

The final product of this section should be a representative estimate of remaining life and a final set of E-values for the existing pavement that can be used with reasonable confidence in the mechanistic model for overlay thickness design in Section A12.

SECTION A9: OVERLAY MATERIALS AND TECHNIQUES

Two basic overlay materials are used in overlay design, i.e., asphaltic concrete (AC) and portland cement concrete (PCC).

PORTLAND CEMENT CONCRETE OVERLAYS

Two basic overlay techniques are available when PCC overlays are placed. They are (1) unbonded PCC overlays, and (2) thin bonded overlays.

Unbonded PCC Overlays

A bond breaker (usually a low stiffness asphaltic concrete, 1 to 2 inches thick) is placed between the old pavement and the new PCC overlay to prevent reflective cracking. Unbonded PCC overlays have been placed successfully on existing pavements with very low remaining life values and this type of overlay is thus very flexible in its application. However, existing slabs that are rocking, severely pumping or faulted, should be stabilized by techniques such as undersealing prior to overlay to avoid severe stress concentrations in the overlay. The preparation of the existing pavement prior to overlay placement, is discussed in more detail in Section A11. A minimum PCC overlay thickness of 6 inches is recommended. Since there will be a considerable vertical height increase, additional design and cost considerations such as impairment of vertical clearance under structures, disruption of and need for alteration of existing drainage patterns, and the need to increase the height of railings and barriers, should be considered. The construction of rigid or flexible shoulders must also be considered.

Thin Bonded PCC Overlays

A thin bonded PCC overlay generally has a minimum thickness of 2 inches for CRP and 3 inches for JCP. This type of overlay must be bonded to the existing PCC pavement. To ensure an adequate bond, the existing surface should be cleaned of all surface contaminants including oil, paint, and unsound concrete. This can be accomplished by cold milling, sand blasting, water blasting or a combination of the above. A grout made from sand and cement or neat cement should be placed on the cleaned surface just in front of the paver and broomed in. The grout should not be allowed to dry before the overlay is placed. Since all cracks in the old surface will reflect through the overlay, all joints in the original pavement must be reproduced in the overlay. For this reason, thin concrete overlays should be used only when the existing concrete is in good condition and surface corrections are necessary. A minimum of 10 to 15 percent remaining life of the existing pavement is recommended. This overlay type is particularly cost-effective when very high traffic growth rates are encountered and avoid excessive user delay costs that will be inevitable if the overlay placement is delayed. The lower tensile stress in the existing PCC slab as a result of this overlay can extend the life of the existing pavement significantly.

ASPHALTIC CONCRETE OVERLAYS

Because of the nature of AC materials, an AC overlay will always be "bonded" to the underlying layer. AC overlays have been placed successfully in existing pavement which both high and very low remaining live values.

Since the principal causes of cracking in an AC overlay are thermal contractions and expansions, and vertical differential deflections of the underlying slabs, some effort must be made to mitigate these stresses. Differential deflections at cracks or joints are considered to be more critical due to the quicker loading rate. If excessive, this vertical deflection can be reduced by undersealing and breaking or replacement of slabs. For horizontal movements a crack relief layer, stress absorbing membrane, or a fabric membrane interlayer could be utilized. The recommended thickness ranges for AC overlay are 2 to 8 inches and 4 to 8 inches when the existing pavement is CRCP and JCP respectively. Additional design and cost considerations, similar to that of unbonded PCC overlays will apply in the case of thick AC overlays.

SHOULDER CONSTRUCTION

The presence and type of shoulder directly affects the stress condition in the PCC slab. The addition of shoulder alone at the appropriate time can thus extend the remaining structural life of the existing pavement. A remaining life values lower than 10 percent, should construction is, however, normally combined with overlay placement.

TECHNIQUES TO MINIMIZE REFLECTION CRACKING

Past experience has shown that it is not possible to design an overlay so that reflection cracking will be completely eliminated. It is possible, however, to design one so that reflection cracking will be minimized. Some of the techniques available for minimizing reflection cracking include (in no specific order):

(1) increased overlay thickness,
(2) placement of an intermediate or cushion layer prior to overlay,
(3) placement of a bond breaker,
(4) placement of high tensile strength fabric as a stress relieving layer,
(5) placement of wire or other type reinforcement along with the overlay,
(6) pavement undersealing at joints (or cracks),
(7) use of softer asphalt or rubber-asphalt in the paving mix, and
(8) pavement breaking prior to overlay placement.

Reference All deals with this aspect in more detail.

OVERLAY MATERIAL PROPERTIES

Table A9.1 summarizes the properties of typical overlay materials in Texas.

TABLE A9.1 PROPERTIES OF TYPICAL OVERLAY MATERIALS

Overlay material	Elastic modulus (E-value) PSI	Poisson's ratio (μ)	Concrete flexural strength (f_c) PSI
(a) PCC overlays	3.5×10^6-7×10^6	0.15	400-800
Coarse aggregate type:			
- River gravel	5×10^6-7×10^6	0.15	500-800
- Lime stone	3.5×10^6-5×10^6	0.15	400-550
(b) AC overlays	300 000-500 000	0.35	-
(c) Intermediate layers (for unbonded PCC)			
(i) Bituminous material	80 000-150 000	0.35	-
(ii) High quality unbound granular material	25 000-50 000	0.40	-

SECTION A10: DESIGN STRATEGIES

SELECTION OF CANDIDATE STRATEGIES FOR FURTHER EVALUATION

Variables such as different overlay types and techniques, timing of the overlay(s), extended maintenance options, etc. can potentially generate a vast number of alternative overlay strategies. It is obviously not possible to evaluate all these strategies in the process of selecting the optimal strategy. The main objective of this section is to present guidelines to assist the design engineer in the selection of a reduced set of candidate optimal design strategies. These guidelines are based on (1) sensitivity analysis using the Rigid Pavement Overlay Design System (Ref A5) to evaluate numerous design strategies under different conditions, (2) reflection cracking analysis (Ref A11) and (3) past experience. The guidelines address important aspects of a design strategy such as the likely combination of overlay type and timing of the overlay(s) (as a function of the remaining life of the existing pavement at overlay placement) that will potentially lead to optimal overlay design strategies in terms of cost and performance.

STEPS TO FOLLOW

(1) Use the estimate of remaining life of the existing pavement (Section A8) to enter the decision tree in Fig A10.1.

(2) Follow the branch that applies to the existing pavement to obtain the nature of the repair strategy prior to overlay placement. The specifics of the repair strategy and how it should be reflected in the overlay thickness design analysis (in Section A12) are evaluated further in Section A11.

(3) The branch will lead to a basic set of overlay strategies. The decision tree then refers to a table that will present guidelines and recommended candidate strategies as a function of remaining

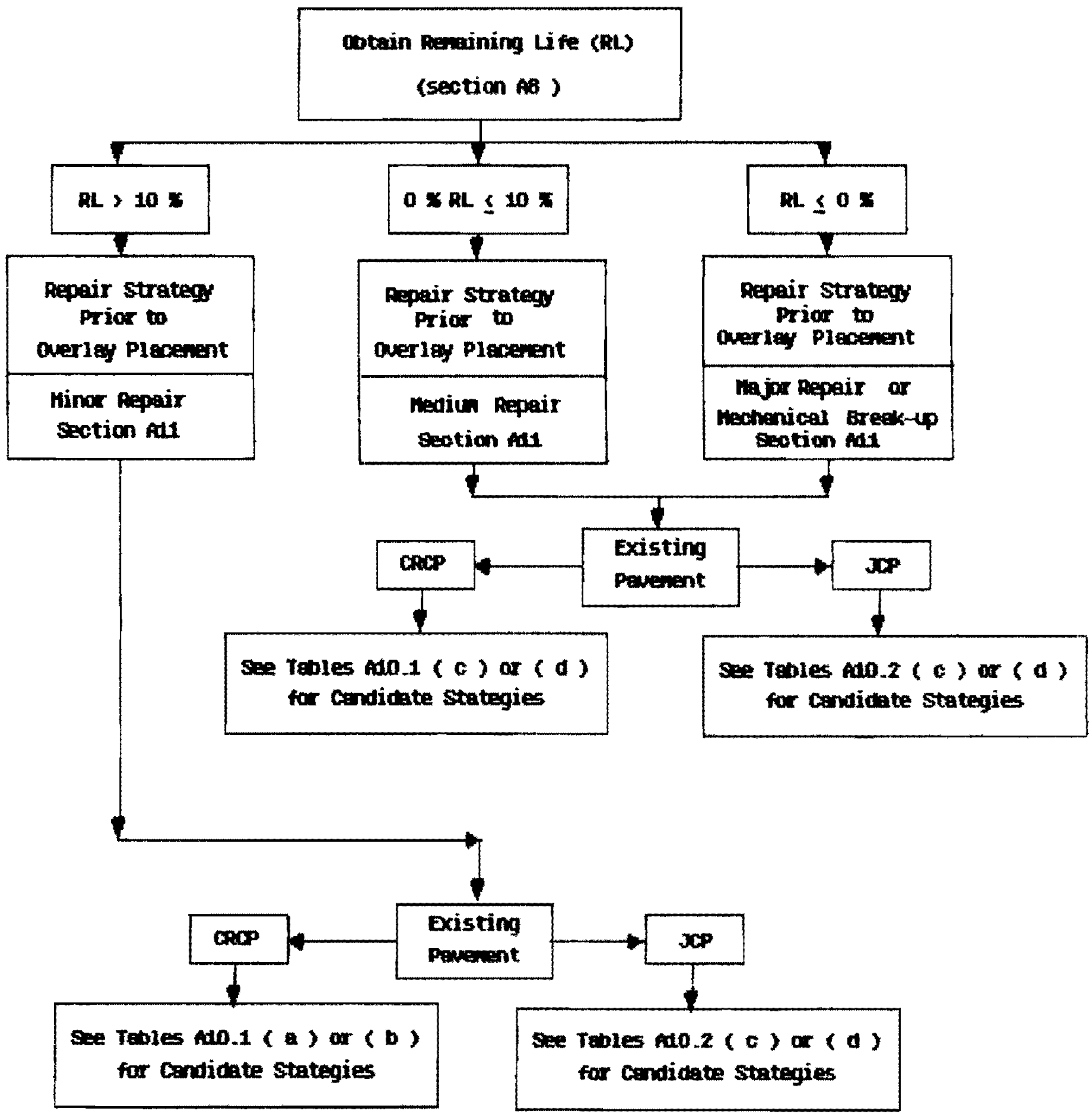


Fig A10.1. Selection of overlay design stragegies.

TABLE A10.1(A). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE **CRCP**

● PRESENT REMAINING LIFE **40 %**

SUBGRADE LEVEL	BASIC OVERLAY TYPE	COST INFORMATION	LOW				MEDIUM				HIGH			
		TRAFFIC LEVEL	LOW				MEDIUM				HIGH			
		AVERAGE DAILY TRAFFIC (EPU'S)**	15000				30000				60000			
		INITIAL YEARLY 18-K ESAL	547500				1095000				2190000			
		REMAINING LIFE AT OVERLAY PLACEM.	40%	20%	10%	0%	40%	20%	10%	0%	40%	20%	10%	0%
LOW 8000 PSI	AC	TOTAL COST RATIO	1.32	1.09	1.05	1	1.10	1	1.06	1.07	2.37	3.35	3.77	4.29
		DELAY COST (%)	1	1	2	1	5	5	8	13	89	93	94	95
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	2.93	2.39	2.18	2.03	1.96	1.77	1.74	1.74	2.55	3.17	3.50	3.95
		DELAY COST (%)	10	11	12	12	17	19	21	25	87	90	91	92
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	3.56	2.87	2.62	2.41	2.34	2.11	2.05	2.04	2.63	3.20	3.57	4.02
		DELAY COST (%)	8	9	10	10	14	16	18	22	84	88	89	91
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	2.12	1.77	1.64	*	1.47	1.39	1.41	*	1.59	2.33	2.60	*
		DELAY COST (%)	11	12	13		19	20	22		82	88.	89	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.27	1.08	1.73	*	1.53	1.41	1.37	*	1	1.55	2.17	*
		DELAY COST (%)	10	12	12		19	20	21		71	82	86	
MEDIUM 14000 PSI	AC	TOTAL COST RATIO	1.45	1.17	1.07	1	1.11	1.03	1	1.01	2.10	2.86	3.39	4.17
		DELAY COST (%)	1	2	2	2	4	5	10	15	89	92	94	95
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	3.43	2.62	2.35	2.12	2.44	2.18	2.16	2.17	2.64	3.51	4.13	5.03
		DELAY COST (%)	10	11	12	13	17	20	26	30	87	91	92	94
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	4.17	3.14	2.80	2.87	2.92	2.57	2.53	2.51	2.71	3.58	4.20	5.10
		DELAY COST (%)	8	10	10	11	40	17	22	26	84	89	91	93
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	2.47	1.95	1.77	*	1.80	1.62	1.57	*	1.31	1.69	2.49	*
		DELAY COST (%)	11	13	13		20	22	24		79	85	89	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.68	2.06	1.86	*	1.89	1.69	1.63	*	1	1.24	1.42	*
		DELAY COST (%)	10	12	13		19	21	23		73	80	82	
HIGH 20000 PSI	AC	TOTAL COST RATIO	1.50	1.18	1.08	1	1.13	1.02	1	1	1.47	2.06	2.50	4.03
		DELAY COST (%)	1.	2	2	2	3	5	10	14	86	91	93	95
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	3.59	2.66	2.35	2.11	2.85	2.53	2.53	2.54	2.67	3.70	4.48	5.78
		DELAY COST (%)	10	10	12	13	17	22	27	32	87	91	93	95
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	4.35	3.18	2.80	2.50	3.41	2.98	2.95	2.91	2.95	3.77	4.55	5.85
		DELAY COST (%)	8	10	10	11	14	18	24	20	84	90	92	94
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	2.57	1.97	1.77	*	2.09	1.86	1.81	*	1	1.77	2.10	*
		DELAY COST (%)	11	13	14		20	22	25		74	85	88	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.74	2.09	1.87	*	2.21	1.95	1.88	*	1.01	1.29	1.51	*
		DELAY COST (%)	11	12	13		19	21	24		73	81	84	

* OVERLAY STRATEGY NOT RECOMMENDED

** TRAFFIC IN BOTH DIRECTIONS

TABLE A10.1(B). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE **CRCP**

● PRESENT REMAINING LIFE **20 %**

Subgrade Level	Basic Overlay Type	Cost Information	Low 40%	Low 20%	Low 10%	Low 0%	Medium 40%	Medium 20%	Medium 10%	Medium 0%	High 40%	High 20%	High 10%	High 0%
Traffic Level			LOW				MEDIUM				HIGH			
Average Daily Traffic (EPU's)			15000				30000				60000			
Initial Yearly 18-K ESAL			547 500				1095 000				2 190 000			
Remaining Life at Overlay Placem.			40%	20 %	10 %	0 %	40%	20%	10%	0 %	40%	20%	10%	0 %
LOW 8000 PSI	AC	TOTAL COST RATIO		1.10	1.02	1		1.09	1.03	1		2	2.24	2.51
		DELAY COST (%)		2	2	2		5	5	5		90	91	92
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		2.15	1.95	1.78		1.78	1.69	1.63		1.91	2.12	2.37
		DELAY COST (%)		9	10	10		17	18	18		87	88	89
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		2.60	2.35	2.13		2.12	2.01	1.93		1.97	2.17	2.40
		DELAY COST (%)		8	8	9		14	15	15		9	86	88
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		1.59	1.46	*		1.44	1.39	*		1	1.44	*
		DELAY COST (%)		11	11			18	19			76	84	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		1.68	1.54	*		1.42	1.36	*		1.23	1.34	*
		DELAY COST (%)		10	11			18	19			80	82	
MEDIUM 14000 PSI	AC	TOTAL COST RATIO		1.20	1.09	1		1.09	1.04	1		2.08	2.42	2.83
		DELAY COST (%)		1	1	1		4	4	6		88	90	92
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		2.75	2.41	2.14		2.16	2.02	1.94		2.61	3	3.47
		DELAY COST (%)		9	10	11		17	18	20		87	89	91
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		3.33	2.89	2.55		2.57	2.40	2.29		2.68	3.07	3.54
		DELAY COST (%)		8	8	9		14	15	17		84	87	89
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		2	1.79	*		1.62	1.55	*		1.63	1.86	*
		DELAY COST (%)		11	12			19	20			82	84	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		2.13	1.90	*		1.70	1.61	*		1	1.49	*
		DELAY COST (%)		10	11			18	19			72	80	
HIGH 20 000 PSI	AC	TOTAL COST RATIO		1.21	1.09	1		1.15	1.06	1		1.45	1.70	2.03
		DELAY COST (%)		1	1	1		2	3	5		85	88	90
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		2.84	2.44	2.15		2.67	2.40	2.39		2.62	3.07	3.64
		DELAY COST (%)		9	10	11		17	18	21		87	89	91
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		3.44	2.93	2.56		3.18	2.94	2.80		2.70	3.14	3.71
		DELAY COST (%)		8	9	9		14	15	16		84	87	89
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		2.06	2.86	*		1.99	1.87	*		1.31	1.89	*
		DELAY COST (%)		11	12			19	20			79	85	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		2.19	1.91	*		2.08	1.95	*		1	1.13	*
		DELAY COST (%)		10	11			18	19			73	76	

* OVERLAY STRATEGY NOT RECOMMENDED

** TRAFFIC IN BOTH DIRECTIONS

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TABLE A10.1(C). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE: CRCP

● PRESENT REMAINING LIFE: 10 %

SUBGRADE LEVEL	BASIC OVERLAY TYPE	COST INFORMATION	LOW				MEDIUM				HIGH			
		AVERAGE DAILY TRAFFIC (EPU'S)	15000				30000				60000			
		INITIAL YEARLY 18-K ESAL	547 500				1095 000				2 190 000			
		REMAINING LIFE AT OVERLAY PLACEM.	40%	20 %	10 %	0 %	40%	20%	10%	0 %	40%	20%	10%	0 %
LOW 8000 PSI	AC	TOTAL COST RATIO			1.07	1			1.04	1			1.63	1.82
		DELAY COST (%)			2	2			5	5			90	91
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			1.93	1.77			1.69	1.61			1.55	1.72
		DELAY COST (%)			9	10			17	17			86	88
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			2.33	2.12			2.01	1.91			1.60	1.76
		DELAY COST (%)			8	8			14	15			84	86
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO			1.35	*			1.45	*			1.36	*
		DELAY COST (%)			11				18				85	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO			1.41	*			1.36	*			1	*
		DELAY COST (%)			10				18				79	
MEDIUM 14000 PSI	AC	TOTAL COST RATIO			1	1			.72	1			1.55	1.81
		DELAY COST (%)			1	1			4	4			88	90
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			2.41	2.13			1.40	1.31			1.95	2.24
		DELAY COST (%)			9	1			17	18			86	89
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			2.91	2.45			1.66	1.56			2.01	2.29
		DELAY COST (%)			8	8			14	15			84	86
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO			1.61	*			1.07	*			1.24	*
		DELAY COST (%)			11				19				8	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO			1.69	*			1.11	*			1	*
		DELAY COST (%)			11				18				77	
HIGH 20 000 PSI	AC	TOTAL COST RATIO			1.10	1			1.06	1			1.43	1.68
		DELAY COST (%)			1	1			3	3			85	88
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			2.51	2.18			2.31	2.15			2.58	3.03
		DELAY COST (%)			9	10			17	18			86	89
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			3.04	2.61			2.75	2.55			2.66	3.16
		DELAY COST (%)			10	9			14	15			84	87
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO			1.64	*			1.74	*			1.62	*
		DELAY COST (%)			11				19				82	
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO			1.73	*			1.82	*			72	
		DELAY COST (%)			11				18				72	

* OVERLAY STRATEGY NOT RECOMMENDED

** TRAFFIC IN BOTH DIRECTIONS

TABLE A10.1(D). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE: CRCP

● PRESENT REMAINING LIFE: 0 %

Subgrade Level	Basic Overlay Type	Cost Information	Traffic Level: LOW				MEDIUM				HIGH			
		Average Daily Traffic (EPU's)	15000				30000				60000			
		Initial Yearly 18-K ESAL	547 500				1095 000				2 190 000			
		Remaining Life at Overlay Placem.	40%	20 %	10 %	0 %	40%	20%	10%	0 %	40%	20%	10%	0 %
LOW 8000 PSI	AC	TOTAL COST RATIO				1				1				**
		DELAY COST (%)				2				5				
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				1.53				1.50				1.09
		DELAY COST (%)				9				16				87
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				1.84				1.77				1
		DELAY COST (%)				8				14				83
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
MEDIUM 14000 PSI	AC	TOTAL COST RATIO				1				1				1
		DELAY COST (%)				1				4				89
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				1.81				1.72				1.10
		DELAY COST (%)				9				17				85
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				2.19				2.03				1.12
		DELAY COST (%)				8				14				84
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
HIGH 20 000 PSI	AC	TOTAL COST RATIO				1				1				1
		DELAY COST (%)				1				4				86
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				2.19				1.95				1.47
		DELAY COST (%)				9				17				86
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				2.64				2.31				1.51
		DELAY COST (%)				8				14				84
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												

** EXCESSIVE AC-OVERLAY THICKNESS

* OVERLAY STRATEGY NOT RECOMMENDED

*** TRAFFIC IN BOTH DIRECTIONS

TABLE A10.2(A). ALTERNATIVE OVERLAY STRATEGIES

- EXISTING PAVEMENT TYPE: JCP
- PRESENT REMAINING LIFE: 40%

SUBGRADE LEVEL	BASIC OVERLAY TYPE	COST INFORMATION	LOW				MEDIUM				HIGH			
		AVERAGE DAILY TRAFFIC (EPU's)	15000				30000				60000			
		INITIAL YEARLY 18-K ESAL	547500				1095000				2190000			
		REMAINING LIFE AT OVERLAY PLACEM.	40%	20%	10%	0%	40%	20%	10%	0%	40%	20%	10%	0%
LOW 8000 PSI	AC	TOTAL COST RATIO	1.35	1.13	1.05	1	1.07	1	1.03	1.05	1.81	2.82	3.39	4.25
		DELAY COST (%)	2	2	2	2	4	7	13	18	88	93	94	95
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	2.50	1.97	1.79	1.66	1.78	1.62	1.64	1.64	2.05	2.78	3.31	4.11
		DELAY COST (%)	9	11	11	11	17	20	25	30	87	91	93	94
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	3.09	2.38	2.15	1.97	2.16	1.93	1.92	1.91	2.11	2.83	3.37	4.16
		DELAY COST (%)	7	9	9	10	14	17	21	26	84	89	91	93
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	1.98	1.61	*	*	1.43	1.31	*	*	1	1.61	*	*
		DELAY COST (%)	10	11			18	20			76	66.		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.24	1.79	*	*	1.59	1.44	*	*	1	1.01	*	*
		DELAY COST (%)	9	10			16	18			76	82		
MEDIUM 14000 PSI	AC	TOTAL COST RATIO	1.43	1.15	1.05	1	1.11	1	1	1.03	1.60	2.53	3.47	6.29
		DELAY COST (%)	2	2	2	2	3	8	14	20	87	93	95	97
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	2.71	2	1.77	1.64	2.13	1.94	1.96	2.03	2.11	3.26	4.40	7.95
		DELAY COST (%)	9	11	11	11	17	24	31	37	87	92	95	97
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	3.35	2.40	2.10	1.89	2.58	2.29	2.27	2.31	2.18	3.32	4.45	8.01
		DELAY COST (%)	7	9	10	10	14	20	26	33	84	91	93	97
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	2.13	1.64	*	*	1.69	1.52	*	*	1	1.46	*	*
		DELAY COST (%)	10	11			18	23			78	86		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.41	1.89	*	*	1.89	1.68	*	*	1.03	1.48	*	*
		DELAY COST (%)	9	10			16	20			76	63		
HIGH 20000 PSI	AC	TOTAL COST RATIO	1.45	1.15	1.05	1	1.07	1	1.01	1.06	1.36	1.87	2.81	**
		DELAY COST (%)	2	2	2	2	3	10	17	24	86	91	94	
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO	2.78	1.93	1.75	1.59	2.14	1.97	2.01	2.11	2.11	3.52	5.29	**
		DELAY COST (%)	9	11	11	11	17	26	34	41	87	93	96	
	UNB PCC PCC SHOULDER	TOTAL COST RATIO	3.44	2.37	2.06	1.84	2.60	2.31	2.31	2.38	2.18	3.58	5.34	**
		DELAY COST (%)	7	9	10	20	14	22	29	36	84	91	95	
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO	2.18	1.63	*	*	1.70	1.53	*	*	1	1.03	*	*
		DELAY COST (%)	10	12			18	24			78	76		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO	2.47	1.81	*	*	1.90	1.68	*	*	1.55	1.58	*	*
		DELAY COST (%)	9	10			16	21			87	86		

** EXCESSIVE USER DELAY COSTS

* OVERLAY STRATEGY NOT RECOMMENDED

*** TRAFFIC IN BOTH DIRECTIONS

TABLE A10.2(B). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE: JCP

● PRESENT REMAINING LIFE: 20 %

SUBGRADE LEVEL	BASIC OVERLAY TYPE	COST INFORMATION	LOW				MEDIUM				HIGH			
		AVERAGE DAILY TRAFFIC (EPU's)	15000				30000				60000			
		INITIAL YEARLY 18-K ESAL	547 500				1095 000				2190 000			
		REMAINING LIFE AT OVERLAY PLACEM.	40%	20%	10%	0%	40%	20%	10%	0%	40%	20%	10%	0%
LOW 8000 PSI	AC	TOTAL COST RATIO		1.02	1.02	1		1.02	1	1.01		2.03	2.38	2
		DELAY COST (%)		1	1	1		5	5	7		89	91	93
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		1.84	1.72	1.60		1.55	1.5	1.49		2.04	2.37	2.77
		DELAY COST (%)		9	10	10		17	17	19		87	89	91
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		1.39	2.07	1.90		1.89	1.8	1.76		2.10	2.43	2.83
		DELAY COST (%)		7	8	8		14	14	16		84	87	89
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		1.47	*	*		1.26	*	*		1.24	*	*
		DELAY COST (%)		10				18				79		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		1.66	*	*		1.40	*	*		1	*	*
		DELAY COST (%)		9				16				76		
MEDIUM 14000 PSI	AC	TOTAL COST RATIO		1.01	1	1.01		1	1	1.03		1.58	1.97	2.51
		DELAY COST (%)		2	1	1		4	4	9		87	90	92
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		1.88	1.71	1.60		1.81	1.73	1.75		2.08	2.14	3.23
		DELAY COST (%)		9	10	2		17	17	23		87	90	92
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		2.34	2.05	1.87		2.20	2.07	2.05		2.15	2.62	3.29
		DELAY COST (%)		7	8	8		14	14	19		84	88	90
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		1.50	*	*		1.45	*	*		1	*	*
		DELAY COST (%)		10				18				77		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		1.70	*	*		1.53	*	*		1	*	*
		DELAY COST (%)		9				19				76		
HIGH 20 000 PSI	AC	TOTAL COST RATIO		1	1.09	1		1	1	1.06		1.35	1.74	2.30
		DELAY COST (%)		2	1	1		3	3	9		86	89	92
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO		1.9	1.7	1.57		1.99	1.89	1.95		2.11	2.68	3.52
		DELAY COST (%)		9	10	10		17	18	24		87	90	93
	UNB PCC PCC SHOULDER	TOTAL COST RATIO		2.35	2.05	1.83		2.45	2.25	2.27		2.17	2.74	3.58
		DELAY COST (%)		7	8	8		14	15	21		84	88	91
	BONDED PCC FLEX SHOULDER	TOTAL COST RATIO		1.51	*	*		1.58	*	*		1	*	*
		DELAY COST (%)		10				18				78		
	BONDED PCC PCC SHOULDER	TOTAL COST RATIO		1.70	*	*		1.77	*	*		1.03	*	*
		DELAY COST (%)		9				16				77		

* OVERLAY STRATEGY NOT RECOMMENDED

** TRAFFIC IN BOTH DIRECTIONS

TABLE A10.2(C). ALTERNATIVE OVERLAY STRATEGIES

- EXISTING PAVEMENT TYPE: JCP
- PRESENT REMAINING LIFE: 10 %

Subgrade Level	Basic Overlay Type	Cost Information	LOW 40%	LOW 20%	LOW 10%	LOW 0%	MEDIUM 40%	MEDIUM 20%	MEDIUM 10%	MEDIUM 0%	HIGH 40%	HIGH 20%	HIGH 10%	HIGH 0%
		Traffic Level	LOW				MEDIUM				HIGH			
		Average Daily Traffic (EPU's)	15000				30000				60000			
		Initial Yearly 18-K ESAL	547 500				1095 000				2 190 000			
		Remaining Life at Overlay Placem.	40%	20 %	10 %	0 %	40%	20%	10 %	0 %	40%	20%	10 %	0 %
LOW 8000 PSI	AC	TOTAL COST RATIO			1	1.15			1	1.07			1	1.18
		DELAY COST (%)			2	2			5	4			89	90
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			1.66	1.71			1.51	1.53			1	1.18
		DELAY COST (%)			9	9			17	16			87	88
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			2.05	2.03			1.84	1.82			1.04	1.20
		DELAY COST (%)			7	7			14	14			84	86
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												
MEDIUM 14000 PSI	AC	TOTAL COST RATIO			1	1.21			1	1.09			1	1.25
		DELAY COST (%)			2	14			4	3			87	90
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			1.86	1.89			1.74	1.78			1.31	1.62
		DELAY COST (%)			9	8			17	16			87	89
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			1.92	1.94			2.12	3.25			1.35	1.66
		DELAY COST (%)			9	8			14	13			84	87
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												
HIGH 20 000 PSI	AC	TOTAL COST RATIO			1	1.26			1	1.13			1	1.30
		DELAY COST (%)			2	1			3	3			86	88
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO			1.86	1.96			1.96	1.99			1.56	1.99
		DELAY COST (%)			9	8			17	17			87	89
	UNB PCC PCC SHOULDER	TOTAL COST RATIO			2.33	2.28							1.61	2.04
		DELAY COST (%)			7	7							84	87
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO			*	*			*	*			*	*
		DELAY COST (%)												

* OVERLAY STRATEGY NOT RECOMMENDED

TABLE A10.2(D). ALTERNATIVE OVERLAY STRATEGIES

● EXISTING PAVEMENT TYPE: JCP

● PRESENT REMAINING LIFE: 0 %

Subgrade Level	Basic Overlay Type	Cost Information	LOW				MEDIUM				HIGH			
		Average Daily Traffic (EPU's)	15000				30000				60000			
		Initial Yearly 18-K ESAL	547 500				1 095 000				2 190 000			
		Remaining Life at Overlay Placem.	40%	20%	10%	0%	40%	20%	10%	0%	40%	20%	10%	0%
LOW 8000 PSI	AC	TOTAL COST RATIO				1				1				1
		DELAY COST (%)				2				4				89
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				1.61				1.43				1
		DELAY COST (%)				9				17				87
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				1.99				1.73				1.04
		DELAY COST (%)				7				4				84
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
MEDIUM 14000 PSI	AC	TOTAL COST RATIO				1				1				1
		DELAY COST (%)				1				3				86
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				1.84				1.66				1.30
		DELAY COST (%)				9				17				87
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				2.27				2.02				1.35
		DELAY COST (%)				7				14				84
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
HIGH 20 000 PSI	AC	TOTAL COST RATIO				1				1				1
		DELAY COST (%)				2				3				86
	UNB PCC FLEX SHOULDER	TOTAL COST RATIO				1.88				1.82				1.56
		DELAY COST (%)				9				17				87
	UNB PCC PCC SHOULDER	TOTAL COST RATIO				2.23				2.20				1.61
		DELAY COST (%)				7				14				84
	BONDED JCP FLEX SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												
	BONDED JCP PCC SHOULDER	TOTAL COST RATIO				*				*				*
		DELAY COST (%)												

* OVERLAY STRATEGY NOT RECOMMENDED

** TRAFFIC IN BOTH DIRECTIONS

life, traffic level and existing pavement structure. It is very easy to use these tables. Simply enter the appropriate block in the table which corresponds to the specific subgrade and traffic level to identify a set of basic overlay strategies. The cost of the optimal strategy is presented by 1 and the approximate cost ratio of the other strategies relative to the optimum are given. The approximate user delay cost associated with a specific design strategy is presented directly below the cost ratio of the strategy and expressed as percentage of the total cost of the specific strategy.

Because of the many variables involved, cost differences less than 10 percent between strategies are probably not significant and more than one strategy should be evaluated. The tables should thus not be used to select the optimum strategy, but to select a reduced number of candidate strategies for further evaluation.

SECTION A11: SELECTION AND MODELING OF REPAIR STRATEGY

The repair and preparation of the existing pavement prior to overlay placement is an important phase in the overlay design procedure. This section provides guidelines to assist the design engineer in the selection of alternative repair strategies, with special emphasis on the explanation of the input selection to characterize alternative and complementary strategies to overlay placement.

As indicated in previous sections, layer theory analysis assumes that pavement layers are elastic, homogeneous, and isotropic. Finite element theories are used in the design procedure to account for the effect of layer discontinuities or the stresses and strains as calculated by elastic layer theory. These adjustment factors are dependent on the pavement type, overlay type, present condition of the pavement as well as the repair strategy. The repair strategy can be an alternative to overlay placement, such as rigid shoulder construction, or complementary to overlay placement, such as pressure grouting to fill voids underneath the slab, in an effort to reduce stress concentrations and thus overlay thickness. A schematic illustration of how the characteristics of the existing pavement and special analysis techniques can be combined to assist in the selection of the repair strategy and stress adjustment factors is presented in Fig A11.1.

STEPS TO BE FOLLOWED

The following steps are recommended for the selection of the repair strategy and the critical stress factors:

(1) Obtain the remaining life of the existing pavement from Section A8.

(2) Follow the branch in the diagram in Fig A11.1 which corresponds to the remaining life value. The steps in each branch is described in more detail below.

Remaining Life Greater than 10 Percent

Step 1. If condition survey information indicate significant evidence of pumping or loss of load transfer, follow the steps listed in the 0 to 10 percent remaining life branch described in the next paragraph.

Step 2. If condition survey information do not provide significant evidence of pumping or loss of load transfer, repair the areas of localized distress. The cause of the distress, such as localized drainage problems, should also be rectified.

Step 3. If shoulder erosion is the main cause of distress, rigid shoulder construction (tied or untied) can be evaluated as an alternative to overlay placement. The extended structural life provided by shoulder construction must be determined as will be explained in Section A12. It should be clear that this alternative can only be viable when the existing pavement has significant remaining life (>20 percent). The critical stress factors that apply in such an analysis are listed in Table A11.1 as a function of pavement type and shoulder type. Ultimately the decision on

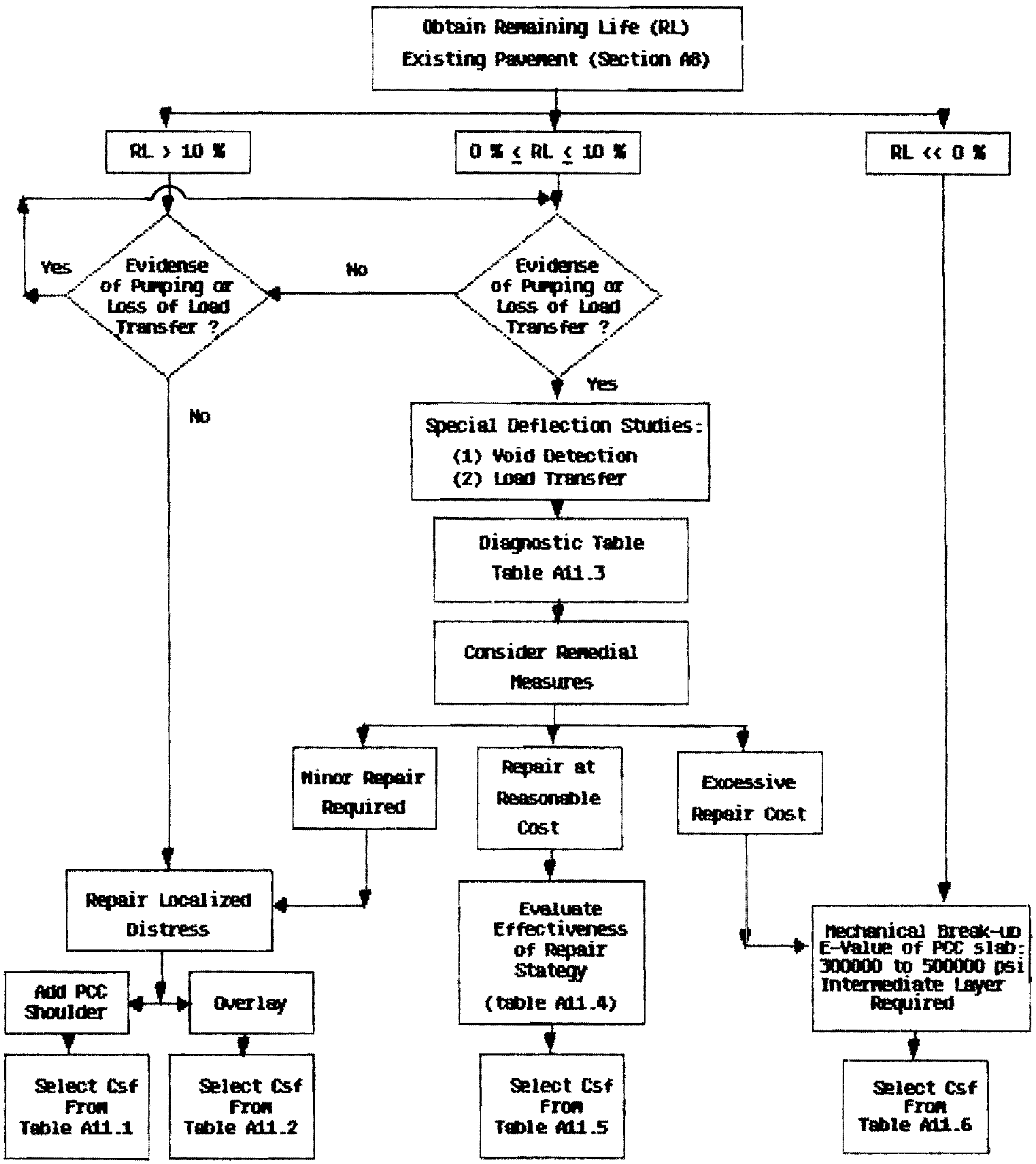


Fig A11.1. Selection of repair strategy and critical stress factor (csf).

TABLE A11.1 EXISTING PAVEMENT CRITICAL STRESS FACTORS FOR DIFFERENT SHOULDER TYPES (REF. A5)

Existing pavement type	Shoulder type	Critical stress factor
CRCP	None	1.25
	AC	1.20
	PCC	1.10
	Tied PCC	1.05
JCP (with load transfer)	None	1.30
	AC	1.25
	PCC	1.20
	Tied PCC	1.10
JCP (without load transfer)	None	1.60
	AC	1.50
	PCC	1.45
	Tied PCC	1.40

which alternative to select, must be based on a cost analysis as described in Section A13.

Step 4. The critical stress factors that apply if an overlay alternative is to be evaluated, are listed in Table A11.2 as a function of pavement type, overlay type, shoulder type and location of the critical stress. The critical layer, i.e., the layer in which the critical stress is located, is the lowest layer in the structure which has remaining life. In other words, if the existing pavement has remaining life, the critical layer will be the existing PCC slab. Once the existing pavement reach the end of its remaining life, the critical stress moves to the first overlay. More information about this design philosophy is provided in Section A12.

Remaining Life from 0 to 10 Percent

Step 1. If condition survey information indicate no evidence of pumping or loss of load transfer, follow the steps listed in the greater than 10 percent remaining life branch as described above.

Step 2. If there is evidence of pumping or loss of load transfer, special deflection studies will be required to determine the extent of the problem. Details regarding studies of this kind are presented in reference A3. Section A3 provides more information on the deflection testing procedures. A short summary of the evaluation techniques is presented in this section.

Step 3. Void Detection. The following basic steps are required to analyze deflection data for void detection:

(1) obtain the outside lane deflection at 1 foot from the pavement outside edge. The sensor 1 deflections are to be corrected for zero temperature differential condition (Ref A3),
(2) obtain inside lane deflection at 3 feet from center line. If the deflection measures are also being made for material characterization at center of the outside lane, this data will be sufficient to provide relative comparison,

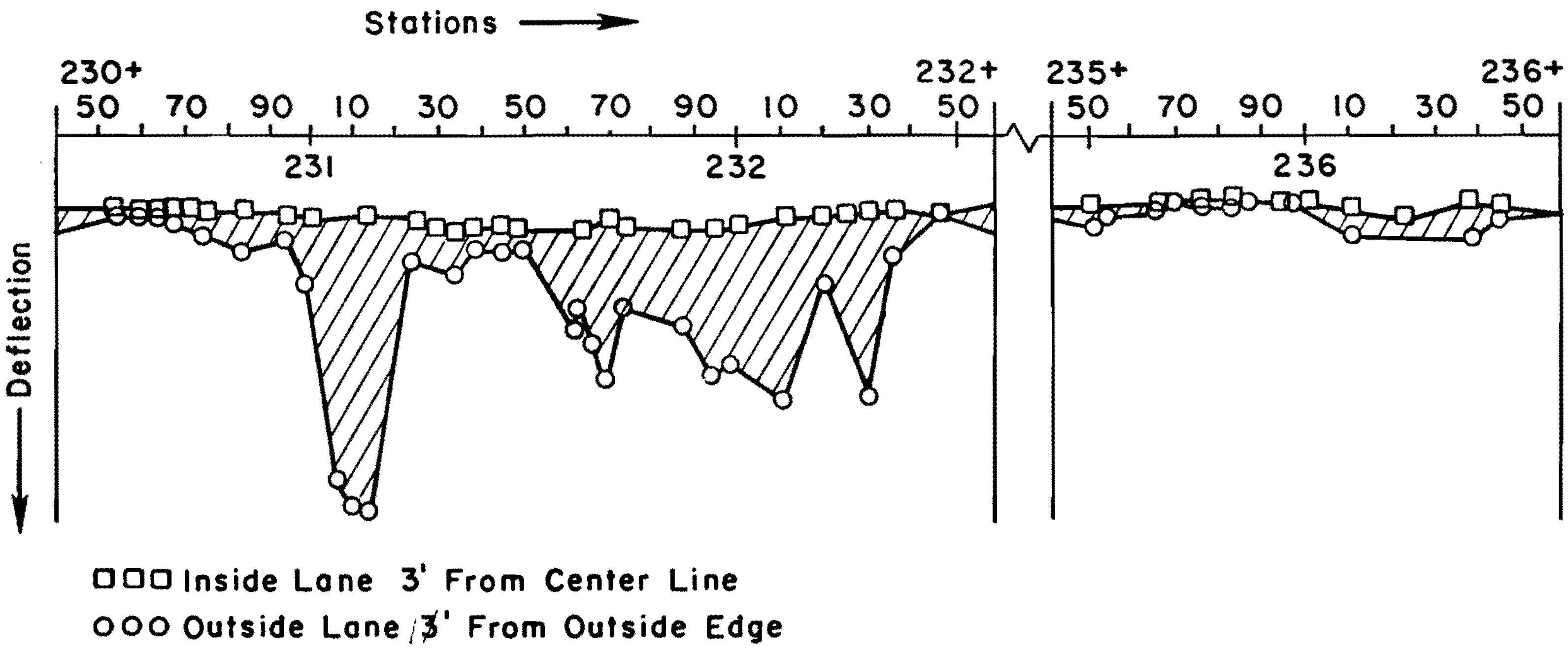


Fig A11.2. Deflection profile of inside and outside lanes (Ref A3).

TABLE A11.2. CRITICAL STRESS FACTORS FOR THE VARIOUS EXISTING PAVEMENT-OVERLAY-SHOULDER COMBINATIONS (REF A5)

Card/ Variable Number	First Overlay Type	Second Overlay Type	Location of Critical Stress	Overlay Shoulder Type	Ratio of Critical to Interior Stress: CRCP Existing Pavement	Ratio of Critical to Interior Stress: JCP Existing Pavement
23.1	ACP	None	Existing Pavement	ACP	1.2	1.4
24.1	ACP	--	Existing Pavement	ACP	1.2	1.4
25.1	ACP	CRCP	Existing Pavement	ACP	1.2	1.3
25.2	ACP	CRCP	Existing Pavement	CRCP	1.05	1.15
26.1	ACP	CRCP	CRCP Overlay	ACP	1.20	1.25
26.2	ACP	CRCP	CRCP Overlay	CRCP	1.05	1.10
27.1	ACP	JCP	Existing Pavement	ACP	1.35	1.40
27.2	ACP	JCP	Existing Pavement	JCP	1.15	1.20
28.1	ACP	JCP	JCP Overlay	ACP	1.35	1.40
28.2	ACP	JCP	JCP Overlay	JCP	1.10	1.15
29.1	Bonded CRCP	None	Existing Pavement	JCP	1.20	--
29.2	Bonded CRCP	None	Existing Pavement	CRCP	1.05	--
30.1	Bonded CRCP	ACP	Existing Pavement	ACP	1.20	--
30.2	Bonded CRCP	ACP	Existing Pavement	CRCP	1.05	--
31.1	Bonded JCP	None	Existing Pavement	ACP	--	1.40
31.2	Bonded JCP	None	Existing Pavement	JCP	--	1.15
32.1	Bonded JCP	ACP	Existing Pavement	ACP	--	1.40
32.2	Bonded JCP	ACP	Existing Pavement	JCP	--	1.15
33.1	Unbonded CRCP	None	Existing Pavement	ACP	1.20	1.30
33.2	Unbonded CRCP	None	Existing Pavement	CRCP	1.05	1.15
34.1	Unbonded CRCP	None	CRCP Overlay	ACP	1.20	1.25
34.2	Unbonded CRCP	Non	CRCP Overlay	CRCP	1.05	1.10
35.1	Unbonded CRCP	ACP	Existing Pavement	ACP	1.20	1.30
35.2	Unbonded CRCP	ACP	Existing Pavement	CRCP	1.05	1.15
36.1	Unbonded CRCP	ACP	CRCP Overlay	ACP	1.20	1.25
36.2	Unbonded CRCP	ACP	CRCP Overlay	CRCP	1.05	1.10
37.1	Unbonded JCP	None	Existing Pavement	ACP	1.35	1.40
37.2	Unbonded JCP	None	Existing Pavement	JCP	1.15	1.20
38.1	Unbonded JCP	None	JCP Overlay	ACP	1.35	1.40
38.2	Unbonded JCP	None	JCP Overlay	JCP	1.10	1.15
39.1	Unbonded JCP	ACP	Existing Pavement	ACP	1.35	1.40
39.2	Unbonded JCP	ACP	Existing Pavement	JCP	1.15	1.20
40.1	Unbonded JCP	ACP	JCP Overlay	ACP	1.35	1.40
40.2	Unbonded JCP	ACP	JCP Overlay	JCP	1.10	1.15

(3) plots of the two deflection profiles are to be produced as illustrated in Fig A11.2, and

(4) areas susceptible to voids are to be marked on the plots on a relative basis as shown in Ref A12.

Step 4. Load Transfer Evaluation. The ratio ($D = \frac{d_c}{d_i}$) of Dynaflect sensor 1 deflection taken at the downstream edge of the transverse crack (d_c) to the midspan deflection (d_i) is used to estimate load transfer. The structural condition of the transverse cracks can be diagnosed by referring to Table A11.3.

Step 5. Remedial Measures. Pressure grouting can be used to fill voids underneath the existing slab. The grouting process is the injection by pressure of a cement grout mixture beneath the slab and/or subbase to fill voids while simultaneously producing a thin layer that should improve deflections and resist future pumping action. Pressure grouting can thus be used to fill voids and improve load transfer resulting in lower stresses in the existing slab and for overlay(s). Other remedial measures such as individual slab replacement may also be justified to repair severely distressed localized areas. The extent and nature of the remedial measures required, is a function of the distress condition of the pavement and economic considerations. If the distress is of such a nature that the repair cost prior to overlay placement will be excessive, the mechanical break-up of the existing slab may be a cheaper alternative. The steps followed in such a case will be the same as those for remaining life values $<<$ 0 percent as described in the next paragraph. When the special deflection studies indicate no significant void or loss of load transfer problem, steps 2 to 4 of the previous paragraph apply. When void and poor load transfer are present and can be repaired at a reasonable cost, this should be done.

Step 6. Effectiveness of Grouting Process. Dynaflect deflections are also used to evaluate the effectiveness of grouting operations to fill voids under the pavement. Practical examples ad a graphical procedure are presented in Ref A12. A step-by-step procedure is presented below.

TABLE A11.3 DIAGNOSTIC CHECKING FOR LOAD TRANSFER AT TRANSVERSE CRACKS ON RIGID PAVEMENT (REF.A3).

Dynaflect* deflection ratio d_c/d_i	Rating for load transfer
1.00 - 1.11	Excellent
1.11 - 1.38	Good to fair
1.38 - 1.54	Fair to poor
1.54 - 1.59	Poor
> 1.60	**Special attention is needed

*Only sensor 1 deflection is required for calculations

**Problem areas to be further investigated.

(1) d_c may be high due to low temperature condition; repeat deflection measurement at crack, d_c at a later time.

(2) Pumping or soft subgrade.

(3) Voids and partial loss of support.

(1) Obtain the Dynaflect deflections after the undersealing operation at 1 foot from the outside edge in the outside lane.

(2) Apply temperature correction to sensor 1 deflections to correspond to zero temperature differential condition (Ref A3).

(3) Plot the corrected deflections before and after the grouting operation as illustrated by dots in Fig A11.3. Also draw the equality line (solid line) which is at 45 degrees with respect to the abscissa.

(4) Using a programmable calculator or statistical package accessible at Texas State Department of Highways and Public Transportation computer, estimate a best fit simple linear regression line (dashed) having its origin in the area of greatest concentration of dots near the line of equality.

(5) Compare the estimated slope of the fitted line, m, with the values shown in Table A11.4 to estimate the effectiveness of the grouting operation.

Step 7. Selection of Critical Stress Factor. The critical stress factors which apply after the remedial measures are presented in Table A11.5.

Remaining Life << 0 Percent

Step 1. The mechanical break-up of the existing slab will probably be necessary since repair cost is likely to be excessive. The E-value of the existing slab after mechanical break-up is in the order of 500,000 psi.

Step 2. The placement of an intermediate layer to limit reflection cracking will be required.

Step 3. Select the critical stress factors that apply to the overlay(s) from Table A11.6.

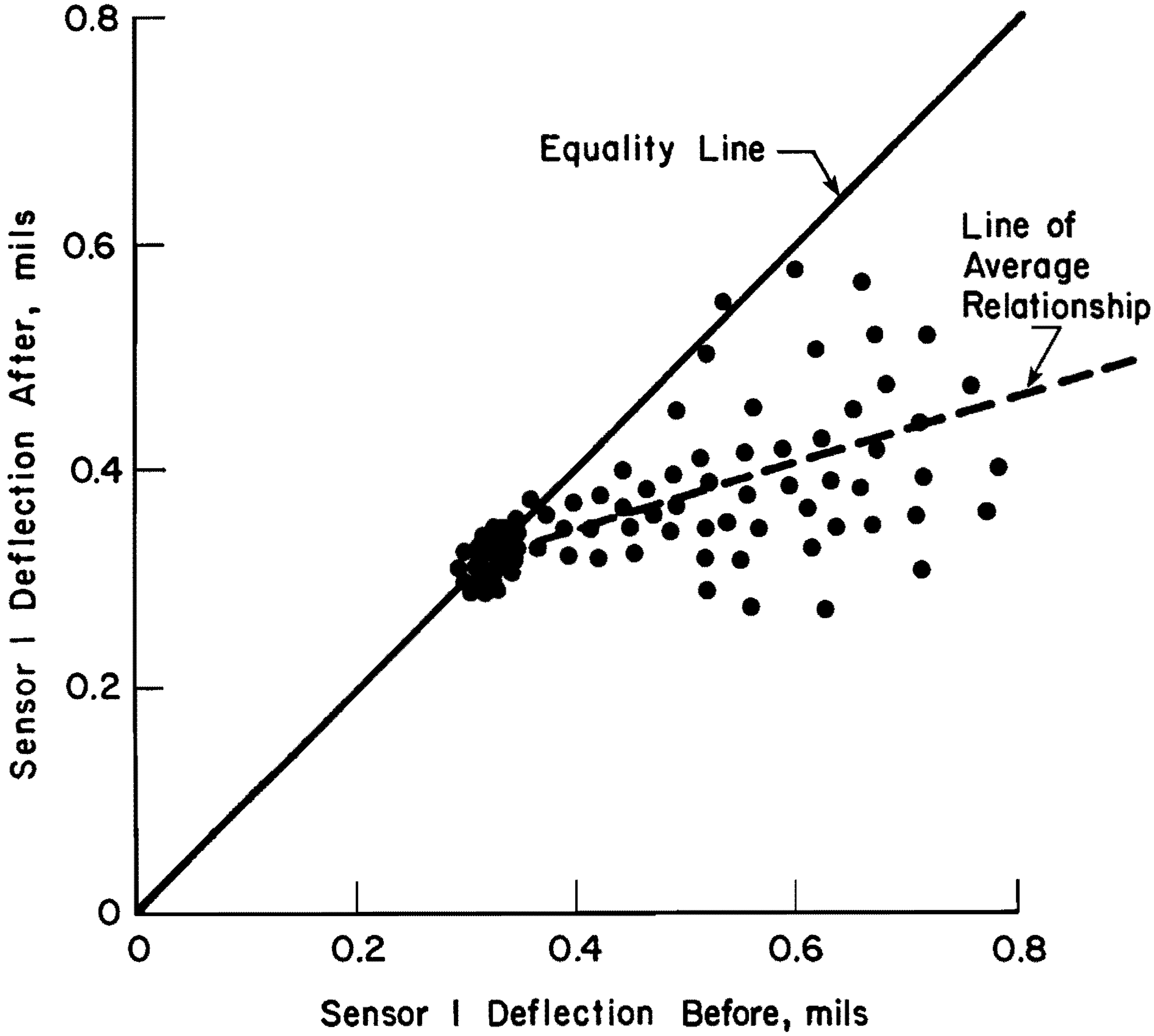


Fig A11.3. Example plot used in the recommended procedure for estimating the effectiveness of undersealing operations (Ref A13).

TABLE A11.4 PERCENT OF VOID AREA FILLED AS A FUNCTION OF SLOPE, m (Ref A13)

m	Percent of void area filled
1.0	0
0.8	20
0.6	40
0.4	60
0.2	80
0.0	100

TABLE A11.5 CRITICAL STRESS FACTORS FOR THE VARIOUS EXISTING PAVEMENT-OVERLAY-SHOULDER COMBINATIONS

					Ratio of Critical to Interior Stress					
					CRCP Existing Pavement			JCP Existing Pavement		
					Percent Void Area Filled					
Card/ Variable Number	First Overlay Type	Second Overlay Type	Location of Critical Stress	Overlay Shoulder Type	100	50	0	100	50	0
23.1	ACP	None	Existing Pavement	ACP	1.20	1.25	1.20	1.40	1.45	1.50
24.1	ACP	--	Existing Pavement	ACP	1.20	1.25	1.30	1.40	1.45	1.50
25.1	ACP	CRCP	Existing Pavement	ACP	1.20	1.25	1.30	1.30	1.35	1.40
25.2	ACP	CRCP	Existing Pavement	CRCP	1.05	1.075	1.10	1.15	1.20	1.25
26.1	ACP	CRCP	CRCP Overlay	ACP	1.20	1.25	1.30	1.25	1.20	1.35
26.2	ACP	CRCP	CRCP Overlay	CRCP	1.05	1.075	1.10	1.10	1.15	1.20
27.1	ACP	JCP	Existing Pavement	ACP	1.35	1.40	1.45	1.40	1.45	1.50
27.2	ACP	JCP	Existing Pavement	JCP	1.15	1.20	1.25	1.20	1.25	1.30
28.1	ACP	JCP	JCP Overlay	ACP	1.35	1.40	1.45	1.40	1.45	1.50
28.2	ACP	JCP	JCP Overlay	JCP	1.10	1.15	1.20	1.15	1.20	1.25
29.1	Bonded CRCP	None	Existing Pavement	JCP	1.20	1.25	1.30	--	--	--
29.2	Bonded CRCP	None	Existing Pavement	CRCP	1.05	1.075	1.10	--	--	--
30.1	Bonded CRCP	ACP	Existing Pavement	ACP	1.20	1.25	1.30	--	--	--
30.2	Bonded CRCP	ACP	Existing Pavement	CRCP	1.05	1.075	1.10	--	--	--
31.1	Bonded JCP	None	Existing Pavement	ACP	--	--	--	1.40	1.45	1.50
31.2	Bonded JCP	None	Existing Pavement	JCP	--	--	--	1.15	1.20	1.25
32.1	Bonded JCP	ACP	Existing Pavement	ACP	--	--	--	1.40	1.45	1.50
32.2	Bonded JCP	ACP	Existing Pavement	JCP	--	--	--	1.15	1.20	1.25
33.1	Unbonded CRCP	None	Existing Pavement	ACP	1.20	1.25	1.30	1.30	1.35	1.40
33.2	Unbonded CRCP	None	Existing Pavement	CRCP	1.05	1.075	1.10	1.15	1.20	1.25
34.1	Unbonded CRCP	None	CRCP Overlay	ACP	1.20	1.25	1.30	1.25	1.30	1.35
34.2	Unbonded CRCP	Non	CRCP Overlay	CRCP	1.05	1.075	1.10	1.10	1.15	1.20
35.1	Unbonded CRCP	ACP	Existing Pavement	ACP	1.20	1.25	1.30	1.30	1.35	1.40
35.2	Unbonded CRCP	ACP	Existing Pavement	CRCP	1.05	1.075	1.10	1.15	1.20	1.25
36.1	Unbonded CRCP	ACP	CRCP Overlay	ACP	1.20	1.25	1.30	1.25	1.30	1.35
36.2	Unbonded CRCP	ACP	CRCP Overlay	CRCP	1.05	1.075	1.10	1.10	1.15	1.20
37.1	Unbonded JCP	None	Existing Pavement	ACP	1.35	1.40	1.45	1.40	1.45	1.50
37.2	Unbonded JCP	None	Existing Pavement	JCP	1.15	1.20	1.25	1.20	1.25	1.30
38.1	Unbonded JCP	None	JCP Overlay	ACP	1.35	1.40	1.45	1.40	1.45	1.50
38.2	Unbonded JCP	None	JCP Overlay	JCP	1.10	1.15	1.20	1.15	1.20	1.25
39.1	Unbonded JCP	ACP	Existing Pavement	ACP	1.35	1.40	1.45	1.40	1.45	1.50
39.2	Unbonded JCP	ACP	Existing Pavement	JCP	1.15	1.20	1.25	1.20	1.25	1.30
40.1	Unbonded JCP	ACP	JCP Overlay	ACP	1.35	1.40	1.45	1.40	1.45	1.50
40.2	Unbonded JCP	ACP	JCP Overlay	JCP	1.10	1.15	1.20	1.15	1.20	1.25

TABLE A11.6. CRITICAL STRESS FACTORS FOR THE VARIOUS EXISTING PAVEMENT-OVERLAY-SHOULDER COMBINATIONS

Card/ Variable Number	First Overlay Type	Second Overlay Type	Location of Critical Stress	Overlay Shoulder Type	Ratio of Critical to Interior Stress	
					CRCP Existing Pavement	JCP Existing Pavement
23.1	ACP	None	Existing Pavement	ACP	1.25	1.45
24.1	ACP	--	Existing Pavement	ACP	1.25	1.45
25.1	ACP	CRCP	Existing Pavement	ACP	1.25	1.35
25.2	ACP	CRCP	Existing Pavement	CRCP	1.075	1.20
26.1	ACP	CRCP	CRCP Overlay	ACP	1.25	1.20
26.2	ACP	CRCP	CRCP Overlay	CRCP	1.075	1.15
27.1	ACP	JCP	Existing Pavement	ACP	1.40	1.45
27.2	ACP	JCP	Existing Pavement	JCP	1.20	1.25
28.1	ACP	JCP	JCP Overlay	ACP	1.40	1.45
28.2	ACP	JCP	JCP Overlay	JCP	1.15	1.20
29.1	Bonded CRCP	None	Existing Pavement	JCP	--	--
29.2	Bonded CRCP	None	Existing Pavement	CRCP	--	--
30.1	Bonded CRCP	ACP	Existing Pavement	ACP	--	--
30.2	Bonded CRCP	ACP	Existing Pavement	CRCP	--	--
31.1	Bonded JCP	None	Existing Pavement	ACP	--	--
31.2	Bonded JCP	None	Existing Pavement	JCP	--	--
32.1	Bonded JCP	ACP	Existing Pavement	ACP	--	--
32.2	Bonded JCP	ACP	Existing Pavement	JCP	--	--
33.1	Unbonded CRCP	None	Existing Pavement	ACP	1.25	1.35
33.2	Unbonded CRCP	None	Existing Pavement	CRCP	1.075	1.20
34.1	Unbonded CRCP	None	CRCP Overlay	ACP	1.25	1.30
34.2	Unbonded CRCP	Non	CRCP Overlay	CRCP	1.075	1.15
35.1	Unbonded CRCP	ACP	Existing Pavement	ACP	1.25	1.35
35.2	Unbonded CRCP	ACP	Existing Pavement	CRCP	1.075	1.20
36.1	Unbonded CRCP	ACP	CRCP Overlay	ACP	1.25	1.30
36.2	Unbonded CRCP	ACP	CRCP Overlay	CRCP	1.075	1.15
37.1	Unbonded JCP	None	Existing Pavement	ACP	1.40	1.45
37.2	Unbonded JCP	None	Existing Pavement	JCP	1.20	1.25
38.1	Unbonded JCP	None	JCP Overlay	ACP	1.40	1.45
38.2	Unbonded JCP	None	JCP Overlay	JCP	1.15	1.20
39.1	Unbonded JCP	ACP	Existing Pavement	ACP	1.40	1.45
39.2	Unbonded JCP	ACP	Existing Pavement	JCP	1.20	1.25
40.1	Unbonded JCP	ACP	JCP Overlay	ACP	1.40	1.45
40.2	Unbonded JCP	ACP	JCP Overlay	JCP	1.15	1.20

SECTION A12: OVERLAY THICKNESS DESIGN (FATIGUE ANALYSIS)

DESIGN CHARTS FOR FATIGUE ANALYSIS

Four design charts which can be used to determine the required overlay thickness for an existing pavement structure consisting of three layers (i.e., PCC slab, subbase and subgrade), are presented in this section. Both rigid and flexible overlays can be considered. When existing structures are more complex, the use of automated solutions (Ref A1 and A5) or elastic layer programs (Ref A4) are recommended. The design philosophy used to develop these charts, is identical to the one being used in the Texas Rigid Pavement Overlay Design System (Ref A5). In order to facilitate the use of the design charts, the design philosophy is summarized below.

Design Philosophy

(1) While the existing PCC slab have remaining life, the critical stress will be located in this layer and the rate of deterioration, i.e., loss of remaining life of the PCC slab will apply for all layers (overlays) above this layer. The existing slab will continue to determine the rate of deterioration while it has remaining life, but the analysis will enter a new phase every time an overlay is placed, since the stress condition in the existing slab will change.

(2) When the existing PCC slab looses all remaining life we enter a new phase in the analysis. The critical response (stress in the case of a PCC overlay strain in the case of an AC overlay) shifts to the first overlay and the material properties of the existing PCC slab change (elastic modulus decrease). The rate of deterioration (loss of remaining life) is now determined by the first overlay. If a second overlay is placed before the first deteriorate to zero remaining life, we enter a new phase in the analysis since the stress condition in the first overlay will change.

(3) When the first overlay reaches zero remaining life, the analysis again enters a new phase. The critical response shifts to the second overlay and the material properties of the first overlay change. The logic of this design philosophy can be followed through to a third overlay.

(4) One exception to this basic philosophy applies when a thin PCC overlay is bonded to the existing slab. In this case the thin bonded PCC overlay reach the end of its life at the same time as the existing PCC slab. Experience has shown that any cracks which exist or originate in the original PCC slab will always propagate up through the bonded PCC overlay.

From the above discussion it is clear that the analysis enters a new phase every time a layer, be it the existing PCC layer or an overlay reaches the end of its life, or when an overlay is placed. The analysis phases can easily be demonstrated by plotting the design strategy against analysis period or design traffic.

The plotting of an overlay strategy is explained by using a typical example.

Details of the strategy are

(1) Analysis period = 20 years
(2) Cumulative future equivalent design traffic N_D = N
(3) Present remaining life of existing PCC slab = 40 percent
(4) Two overlay strategies are plotted in Fig A12.1. Figure A12.1(a) shows the plot of a strategy that requires an unbonded PCC overlay (or an AC overlay when the existing PCC slab reach 20 percent remaining life. Figure A12.1(b) show the plot of a strategy that requires a bonded PCC overlay when the existing PCC slab reaches 20 percent RL;

The delineation of the phases in the analysis follows directly from the discussion in the previous paragraph. The first strategy (unbonded PCC or AC overlay) has three phases, as illustrated in Fig A12.1(a).

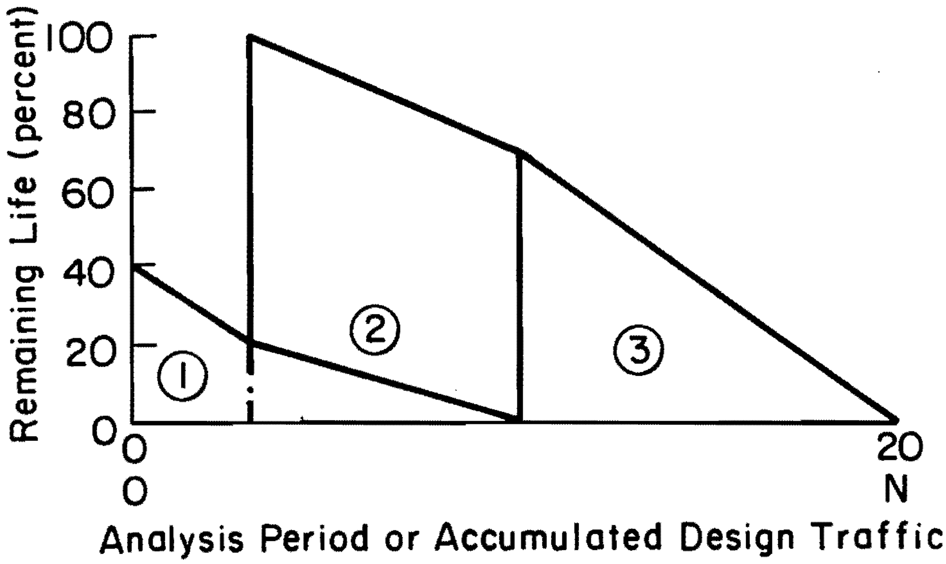


Fig A12.1(a). Design strategy for AC or unbonded PCC overlay.

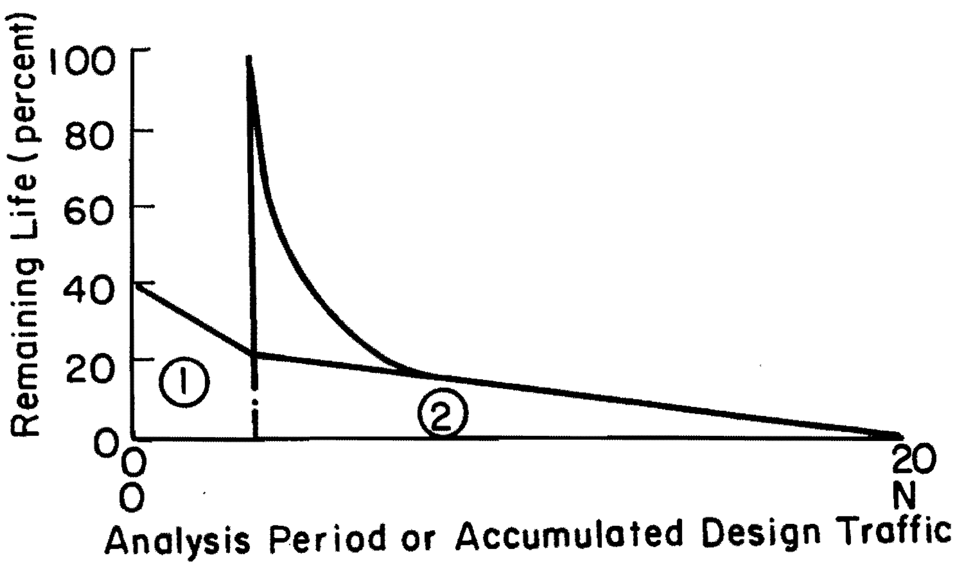


Fig A12.1(b). Design strategy for bonded PCC overlay.

The second strategy (bonded PCC overlay) has 2 phases, as illustrated in Fig A12.1(b).

How to Use Design Charts

Figure A12.2 describes the use of the charts in the form of a flow diagram. The design philosophy and all the steps of the overlay thickness design process are built into the flow diagram. Examples of how to use the individual charts are presented on the chart.

DESIGN CHARTS FOR REFLECTION CRACKING ANALYSIS

Design charts for the design of AC overlays or PCC pavements to prevent reflection cracking is presented by Diaz et al in Ref. A11.

SECTION A13: COST ANALYSIS

Alternative overlay design strategies should be compared on a cost basis. The cost analysis should be regarded as an important aid to decision making. It does not necessarily include all the factors leading to a decision and should therefore not override all other considerations. Factors which are affected directly by highway engineering decisions and can be dollar priced should be included. The main economic factors which determine the cost of a design strategy are the analysis period, the overlay construction cost, the maintenance cost, the road user costs, the salvage value at the end of the analysis period and the real discount rate.

The computer program RPRDS-1 (Ref A5) computes the net present cost value of all the strategies generated by the program based on a comprehensive set of cost related inputs. The strategies are then ranked by the program on a minimum net present cost basis.

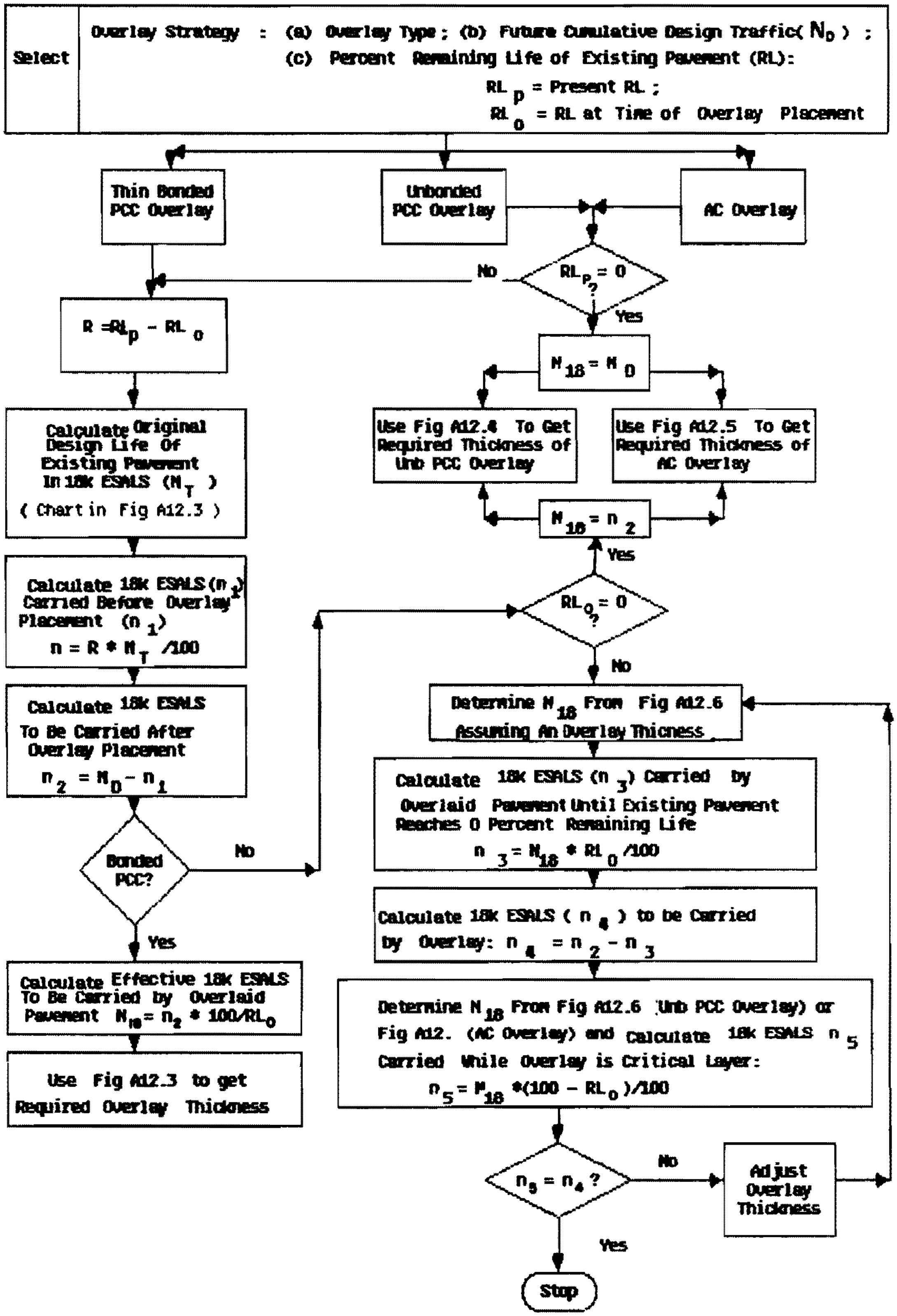


Fig A12.2. Explanation of how to use design charts.

ANALYSIS PERIOD

The period of time over which an analysis is to be made, should be based primarily on the ability of the analyst to forecast the future. This period should normally not exceed 20 years.

OVERLAY CONSTRUCTION COST (C)

The overlay construction cost (C) should be estimated from current contract rates for similar projects in the region. For the purpose of this manual, construction cost include all costs related to construction at the time of overlay placement, i.e.,

(1) mobilization and traffic handling costs;
(2) costs related to the repair and preparation of the existing pavement surface prior to overlay placement;
(3) overlay material and placement cost;
(4) costs related to the construction of shoulders and adjustments to drainage facilities and guard rails, etc.; and
(5) finishing costs, i.e., cleaning and painting.

MAINTENANCE COST (M)

Maintenance costs (M) should include all the costs of maintaining adequate surface integrity, i.e., joint and crack sealing, patching, restoring skid resistance. For the purpose of this manual these costs will exclude surface repair and preparation costs incurred at the time of overlay placement since they will be included under construction costs.

USER DELAY COSTS

For the purpose of this manual user delay costs are broken down into two categories: (1) user delay costs incurred during the period of overlay placement (D) and (2) user delay costs incurred during a maintenance measure (d). Accurate estimates of user delay costs are very difficult to obtain and many factors such as the level of the average daily traffic, the hourly distribution of traffic, time of delay (based on production rates), traffic handling techniques, etc. come into play. The importance of this cost item however, cannot be neglected in the selection of alternative design strategies especially at high traffic levels. At high traffic levels (total traffic > 50,000 equivalent passenger car units) the user delay costs of an overlay strategy can for example be 2 to 5 times the construction cost of the strategy depending on the traffic handling technique. Sophisticated automated delay models are available to estimate delay costs. These models have been incorporated in the computer program RPRDS-1 (Ref A5). Fairly simple calculations can, however, be used for approximate estimates. The Highway Design Manual of the State of Texas (Ref A9) presents guidelines for the estimation of user costs. Tables A10.1 to A10.4 also contain valuable information on the importance of user delay costs under different conditions. These tables can be used for rough estimates if no better information is available. It should be clear from these tables that user delay costs have an overriding impact on both the total strategy cost and timing of the overlay placement at high traffic levels.

SALVAGE VALUE (S)

The salvage value of the overlay at the end of the period under consideration is difficult to assess. If the road is to remain on the same location, the existing pavement layers may have a salvage value, but if the road is to be abandoned at the end of the period under consideration the salvage value could be little or zero. The assessment of salvage value can be approached in a number of ways depending on the method employed to rehabilitate or reconstruct the pavement. A value of 15 to 30 percent of the overlay cost is normally assigned to the overlays. The impact of this decision on the total present worth of cost is, however, not very significant since this cost item is generally discounted over a long period of time.

REAL DISCOUNT RATE

When net present value analysis is done, a real discount rate must be selected to express future expenditure in terms of present day values. The discount rate for performing present value calculations should represent the opportunity cost to the taxpayer, i.e., the estimated average market rate of return that would be achieved if more public transportation funds were left in private hands rather than being paid to the government in taxes.

A range of 6 to 12 percent is common in current economy studies of public projects; for example, the U. S. Office of Management and Budget recommends a 10 percent discount rate for Federal Government economy studies. The possible effects of uniform future price increases (inflation) should be ignored.

Because the results of the net present value calculations are sensitive to the discount rate, the analyst may wish to perform the economic calculations at two or three alternative discount rates. However, all final comparisons of projects should be made using a consistent value of the discount rate.

NET PRESENT VALUE ANALYSIS

The total cost of a project over its life is the sum of construction costs, maintenance costs and user delay costs minus the salvage value. The total cost can be expressed in a number of different ways but, for the purpose of this document, the net present value (NPV) approach has been adopted. The net present value of costs can be calculated as follows:

$$\mathrm{NPV} = \frac{(C_1 + D_1)}{P_{x_1}} + \ldots \frac{(c_i + D_i)}{P_{x_i}} \ldots \rightarrow \text{cost during overlay placement}$$

$$+ \frac{(M_1 + d_1)}{P_{y_1}} + \ldots \frac{(M_i + d_i)}{P_{y_i}} \ldots \rightarrow \text{cost during maintenance activities}$$

where

NPV	=	net present value of cost
C_i	=	construction cost of i^{th} overlay placement
D_i	=	user delay cost during time of i^{th} overlay placement
M_i	=	cost of i^{th} maintenance measure
r	=	real discount rate
d_i	=	user delay cost during i^{th} maintenance measure
x_i	=	number of years from the present to the i^{th} overlay placement (within analysis period)
y_i	=	number of years from the present to the i^{th} maintenance measure (within the analysis period)
z	=	analysis period
s	=	salvage value of the overlays
p	=	discount factor based on the equation $P = (1 + 0.01r)$ A set of discount factors (P) is given in Table A13.1.

All costs must be expressed in terms of current costs. It is normally the easiest to express all costs on a per square yard basis. When the designer is only interested in comparing the cost of two design strategies, only the cost differences need to be included in the analysis.

SECTION A14: DESIGN EXAMPLES

DESIGN INFORMATION

General Information

(1) Four lane facility (2 lanes in each direction)

(2) Location: District 4

(3) Pavement type: CRCP with no PCC shoulder

(4) Pavement age: 6 years

<u>Condition Survey: Section A_2</u>

(1) Distress index: Z_c = 0.5

(2) No visual evidence of pumping or loss of load transfer

<u>Dynaflect Deflection Data: Section A_3</u>

(1) Only one design section considered

(2) Modal sensor 5 deflection (W_{5m}) $\simeq$ 90th percentile sensor 5 deflection = 7 mils * 10^{-1}

(3) Modal deflection slope (W_1 - W_5) modal = 0.000202 inches

<u>Traffic Data: Section A_4</u>

(1) Present yearly equivalent traffic = 2 * 10^6 18k ESAL's in both directions of travel

(2) Estimated future growth rate (i) of equivalent traffic = 3 percent

(3) Cumulative past equivalent traffic (n) = 9.778 *10^6 18k ESAL's in both directions

(4) Average daily traffic in both directions of travel = 55000 ESAL's

(5) Reduction of traffic data. From tables A4.1 and A4.2 the lane distribution factor Le = 0.9 and the directional distribution factor (D) = 0.5. The following reduced traffic information will be required for the design procedure:

(a) Cumulative past traffic in the design lane =

$$n * Le * D = 9.778 * 10^6 * 0.9 * 0.5 = 4.4 * 10^6 \text{ 18k ESAL's;}$$

(b) Future cumulative equivalent traffic in the design lane (N_D).

Using a design period of 20 years, a growth rate (i) of 3 percent to get the traffic growth factor (f) from Table A4.4 it follows that

$$N_D = \text{daily equivalent traffic} * Le * D * f$$

$$= \frac{2 * 10^6}{365} * 0.9 * 0.5 * 10100 = \underset{\text{18k ESAL's}}{24.9 * 10^6}$$

Design Criteria: Section A_5

(1) Time constraints: analysis period = structural design period = 20 years

(2) Traffic constraints: facility must be designed to carry the traffic for the next 20 years

(3) No other constraints

DEFINITION OF DESIGN INPUTS

Selection of Design Section: Section A_6

(1) Only one section considered (refer to Section A6 for procedure)

Structure and Material Properties of Existing Pavement: Section A_7

(1) Construction Records

From the construction records, the following information is available:

(a) CRC-SLAB thickness = 8 inches with elastic modulus (E) $\simeq$ 6 x 10^6 psi

(b) Stabilized subbase thickness = 9 inches

(c) Granular subgrade

(3) Elastic Moduli (E Values)

From deflection data (Section A3) the following information is available:

(a) $W_{5m} \simeq$ W 90th percentile = 7 mils * 10^{-1}

(b) Modal $W_1 - W_5$ = 0.000202 inches

From Figure A7.3, E_3 = subgrade modulus = 8000 psi. Assuming E_1 = 6 * 10^6 psi, the subbase modulus (E_2) can be determined and $E_2 \simeq$ 400,000 psi. These values compare well to similar materials in the region and no adjustments are justified at this stage. The preliminary set of E-values therefore are:

$$E_1 = 6{,}000{,}000 \text{ psi}$$
$$E_2 = 4{,}000{,}000 \text{ psi}$$
$$E_3 = 8{,}000 \text{ psi}$$

(3) Concrete Flexural Strength (f)

The concrete flexural strength is available from laboratory results and f $\simeq$ 650 psi.

Estimation of Remaining Life (RL) and Final Calibration of E-Values: Section A_8

(1) RL_1: RL_1 is the estimate of the remaining life of the existing pavement from a semi-empirical model based on the past behavior of the pavement. The following information is required:

(a) District: D_4

(b) Distress index of design section: Z_c = 0.5

(c) Cumulative past equivalent traffic in design lane (n). From section A4 n 7 * 10^6 18k ESAL's.

(d) Pavement age (A). From design information A < 10 years.

The nomograph in Figure A8.2 can now be used to estimate RL_1 and $RL_1 \simeq$ 36 percent.

(2) $\underline{RL_2}$: RL_2 is the estimate of the remaining life of the existing pavement using the E-value estimates from Section A7 and the mechanistic fatigue model. The following information is required:

(a) Cumulative past equivalent traffic (n) in design lane (i.e. slow lane). From Section A4, $n = 4.4 * 10^6$ 18k ESAL's.

(b) PCC-flexural strength (f). From Section A7, f = 650 psi.

(c) Critical stress factor (csf). From Table A8.1, csf = 1.2.

(d) Original structural design life (N_T) in 18k ESAL's. With the E-values from section A7 and the information above, the design chart in Figure A8.3 can be used to obtain N_T. $N_T = 7.3 * 10^6$ 18k ESAL's.

Using equation A8.1, RL_2 can now be determined.

$$RL_2 = 1 - \frac{n}{N_T} = 1 - \frac{4.4 * 10^6}{7.3 * 10^6} \simeq 40\%$$

(3) <u>Comparison of RL_1 and RL_2</u>

RL_1 and RL_2 compare well and no further adjustments are required.

(4) <u>Final Estimate of RL and E-Values</u>

RL = 40 percent

$E_1 = 6 * 10^6$ psi

$E_2 = 400{,}000$ psi

$E_3 = 8{,}000$ psi

<u>Overlay Materials and Techniques: Section A_9</u>

All overlay materials available in District 4.

Selection of Candidate Strategies for Further Evaluation: Section A_{10}

The following information is required:

(1) Present yearly equivalent traffic in both directions = $2 * 10^6$ 18k ESAL's (from Section A4)

(2) Average daily traffic in both directions = 55000 passenger car units (EPU's)

(3) Remaining life of existing pavement, RL = 40 percent (from Section A8)

With the above information, a candidate set of overlay strategies for further evaluation can be obtained from Table A10.1(a). The two strategies selected are:

(1) Thin bonded PCC-overlay with PCC shoulders placed when possible. The existing pavement has 40 percent remaining life, i.e., overlay as soon as possible. This strategy is likely to be the cheapest and the user delay cost is in the order of 71 percent of the total strategy cost.

(2) Thin bonded PCC overlay with PCC shoulder placed when existing pavement reaches 20 percent remaining life. The user delay cost of this strategy is in the order of 82 percent of the total strategy cost.

Selection and Modeling of Repair Strategy: Section A_{11}

Since the remaining life of the existing pavement is $>>$ 10 percent, and the condition survey information did not show evidence of pumping or loss of load transfer, it follows from Figure A11.1 that

(1) Repair strategy is to repair localized distress, and

(2) Critical stress factor (csf) = 1.05 (from Table A11.2)

<u>Overlay Thickness Design:</u> <u>Section A_{12}</u>

The following is required:

(1) Cumulative equivalent traffic in design lane (N_D). From Section A_4 $N_D = 24.9 * 10^6$ 18_t ESAL's.

(2) E-values: from Section A8, $E_1 = 6 * 10^6$ psi; $E_2 = 400{,}000$ psi and $E_3 = 8{,}000$ psi.

(3) Layer thicknesses of existing pavement, i.e., base = 8 inches and subbase = 9 inches.

(4) RL of existing pavement = 40 percent (from Section A8)

(5) Critical stress factor. From Section A11 csf = 1,05

With the information presented above, the design chart in Figure A12.3 can be used to obtain the required overlay thickness.

<u>Strategy a</u>: Thin bonded PCC overlay with PCC shoulders placed when the existing pavement has 40 percent remaining life. Using the steps in Figure A12.2, it follows that

$RL_p = RL_o = 40$ percent; $R = 0$; $n_1 = 0$; $n_2 = 24.9 * 10^6$ 18k ESAL's, and $N_{18} = n_2 * 100/RL_o$

$$N_{18} = \frac{24.9 * 10^6}{40} * 10$$
$$= 62.25 \text{ 18k ESAL's}$$

From the design chart in Figure A12.3, $D_1' = 12$ inches

The required overlay thickness = $D_1' - D_1$ = 12 inches - 8 inches = 4 inches.

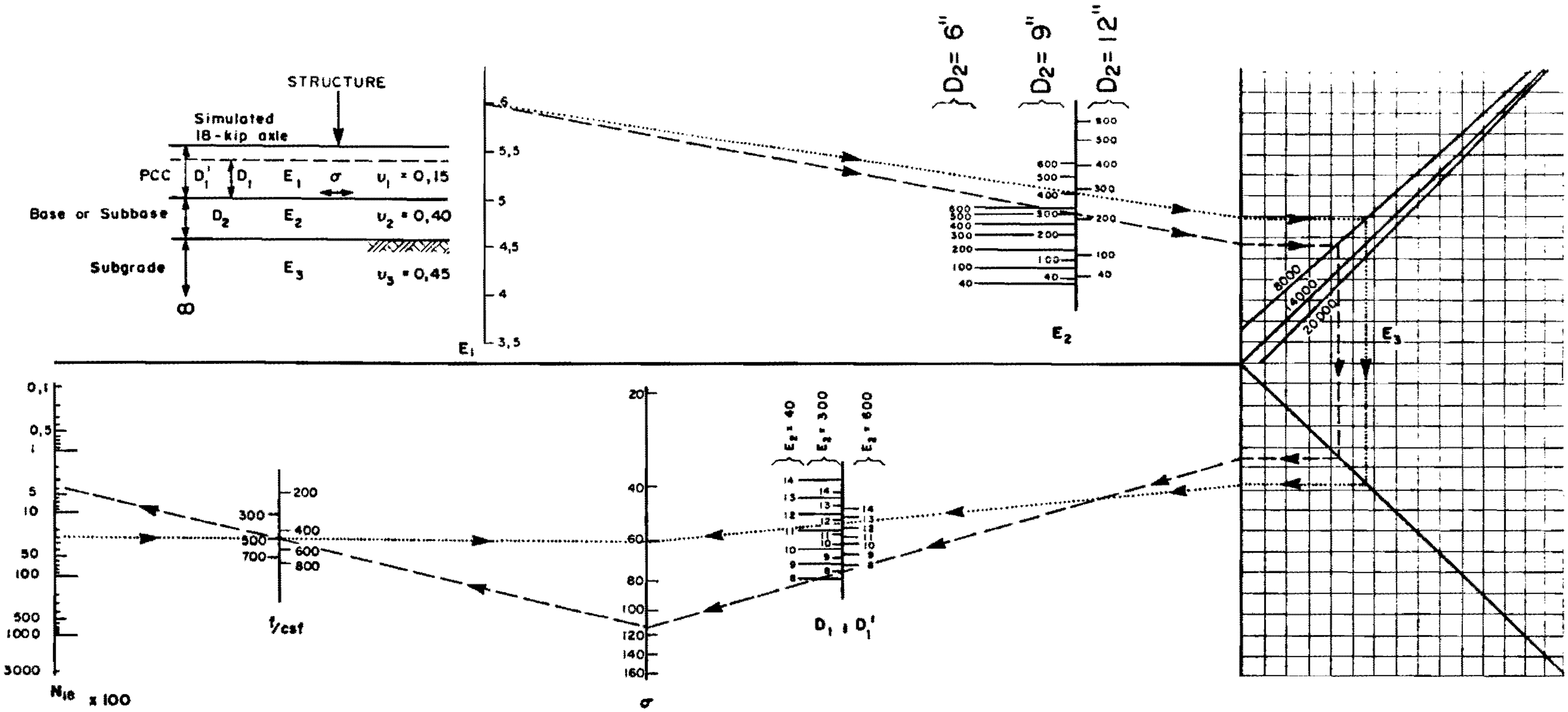
<u>Strategy b</u>: Thin bonded PCC overlay with PCC shoulders placed when existing pavement has 20 percent remaining life. Using the steps in Figure A12.2, it follows that

$R = RL_p - RL_o = 40$ percent $- 20$ percent $= 20$ percent;

$N_T = 7.3 * 10^6$ 18k ESAL's;

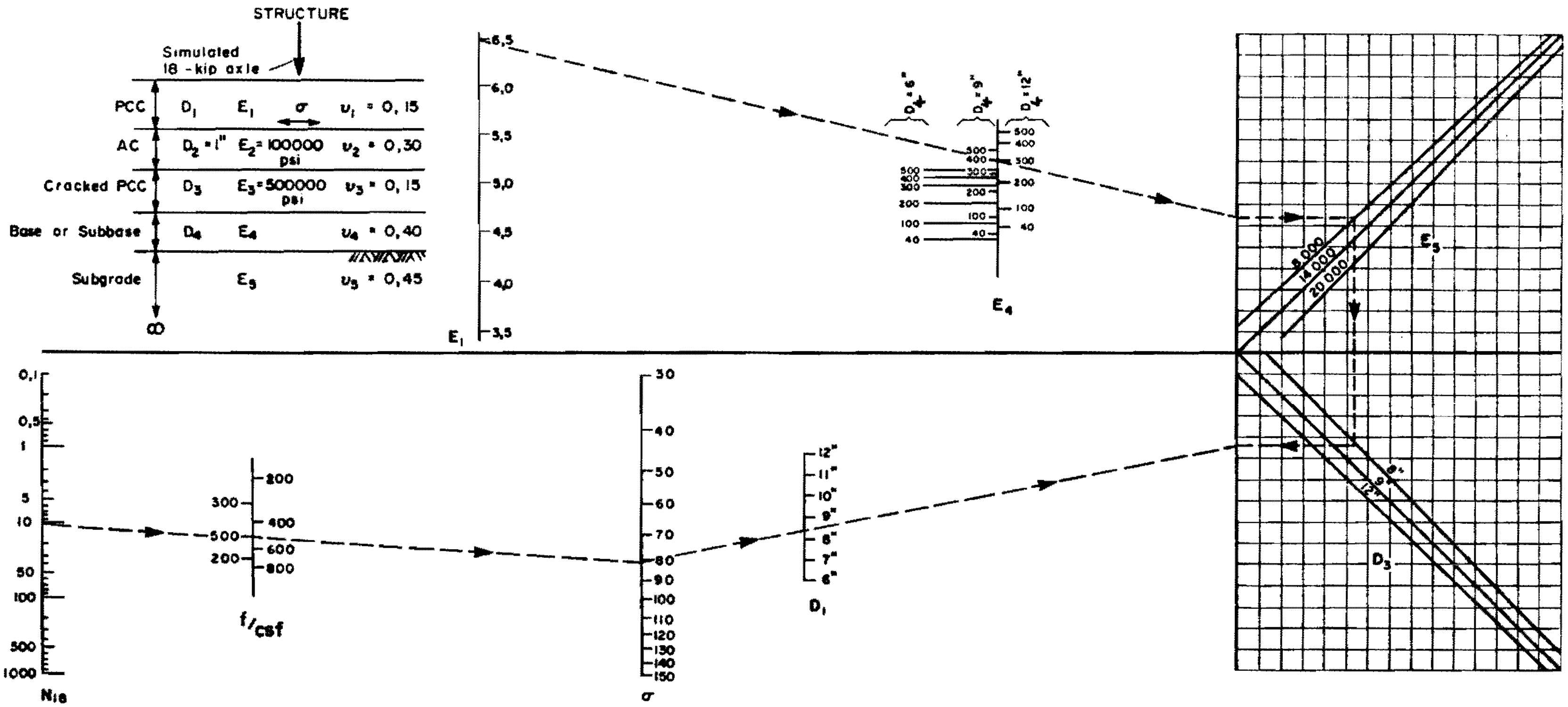
$n_1 = R * N_T/100 = 20 * 7.3 * 10^6/100 = 1.46 * 10^6$ 18 k ESAL's;

$n_2 = N_D - n_1 = 24.9 * 10^6 - 1.46 * 10^6 = 23.44 * 10^6$ 18k



WHERE :	APPLICATION	
N_{18} = NUMBER OF 18-KIP ESAL APPLICATIONS TO TERMINAL DISTRESS CONDITION (10^6 18-KIP ESAL) R.L. = PERCENT REMAINING LIFE OF EXISTING PAVEMENT f = FLEXURAL STRENGTH OF PCC csf = CRITICAL STRESS FACTOR E_1 = PCC - MODULUS (10^6 psi) E_2 = SUBBASE - MODULUS (10^3 psi) E_3 SUBGRADE - MODULUS (psi) D_i = LAYER THICKNESSES (INCHES)	1 CALCULATE ORIGINAL DESIGN LIFE OF EXISTING PAVEMENT (ASSUME RL = 100%)	2. DETERMINE THE REQUIRED THICKNESS OF A THIN BONDED PCC OVERLAY (R.L. OF EXISTING PAVEMENT TO BE SPECIFIED)
	EXAMPLE (a) GIVEN : $E_1 = 6 \times 10^6$ psi ; $E_2 = 300 \times 10^3$ psi ; $E_3 = 8000$ psi $D_1 = 8''$; $D_2 = 9''$; $f = 600$ psi ; CSF = 1,2 (b) REQUIRED: ORIGINAL DESIGN LIFE (c) ANSWER: $N_{18} = 3,9 \times 10^6$ 18 - KIP ESAL (REFER TO BROKEN LINE --------)	EXAMPLE (a) GIVEN : $E_1 = 6 \times 10^6$ psi ; $E_2 = 400 \times 10^3$ psi ; $E_3 = 8000$ psi ; $D_1 = 8''$; $D_2 = 9''$; $f = 600$ psi ; CSF = 1,2 ; RL = 40% (b) REQUIRED: THICKNESS OF THIN BONDED PCC - OVERLAY TO CARRY 10×10^6 18-KIP ESAL (c) ANSWER : $\frac{N}{RL} \times 100 = \frac{10 \times 10^6}{40} \times 100 = 25 \times 10^6$ $D_1' = 12''$ (REFER TO DOTTED LINE ....) $\therefore$ REQUIRED OVERLAY THICKNESS = $D_1' - D_1 = 4''$

Fig A12.3. Chart to: (a) calculate original design life of existing pavement and (b) determine the required thickness of a thin bonded PCC overlay.



WHERE :

N_{18} = NUMBER OF 18-KIP ESAL APPLICATIONS TO TERMINAL DISTRESS CONDITION (10^6 18-KIP ESAL)
f = FLEXURAL STRENGTH OF PCC OVERLAY
CSF = CRITICAL STRESS FACTOR
E_1 = PCC - OVERLAY MODULUS (IN 10^6 psi)
E_2 = BONDBREAKER MODULUS = 100 000 psi
E_3 = EXISTING PCC MODULUS = 500000 psi (EXISTING PCC CRACKED WITH ZERO REMAINING LIFE)
E_4 = SUBBASE MODULUS (IN 10^3 psi)
E_5 = SUBGRADE MODULUS (IN psi)
D_i = LAYER THICKNESSES (IN INCHES)

APPLICATION

DETERMINE THE REQUIRED THICKNESS OF AN UNBONDED PCC - OVERLAY

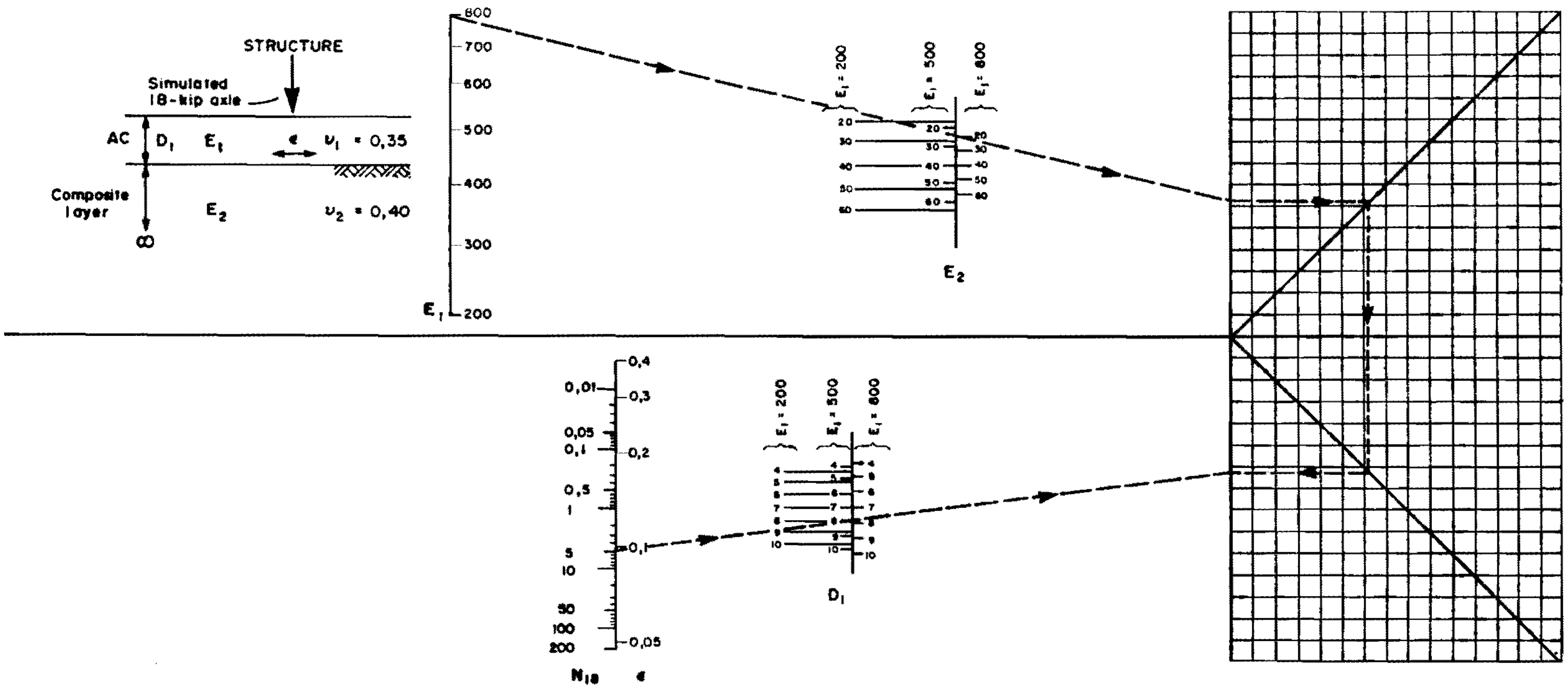
EXAMPLE

(a) GIVEN : E_1 = 6,5 x 10^6 psi ; E_2 = 100000 psi ; E_3 = 500 000 psi ; E_4 = 300 x 10^3 psi ;
E_5 = 8 000 psi ; D_2 = 1" ; D_3 = 8" ; D_4 = 12" ; f=600 ; csf=1.2

(b) REQUIRED : THICKNESS OF UNBONDED PCC - OVERLAY TO CARRY 10 x 10^6 18-KIP ESAL

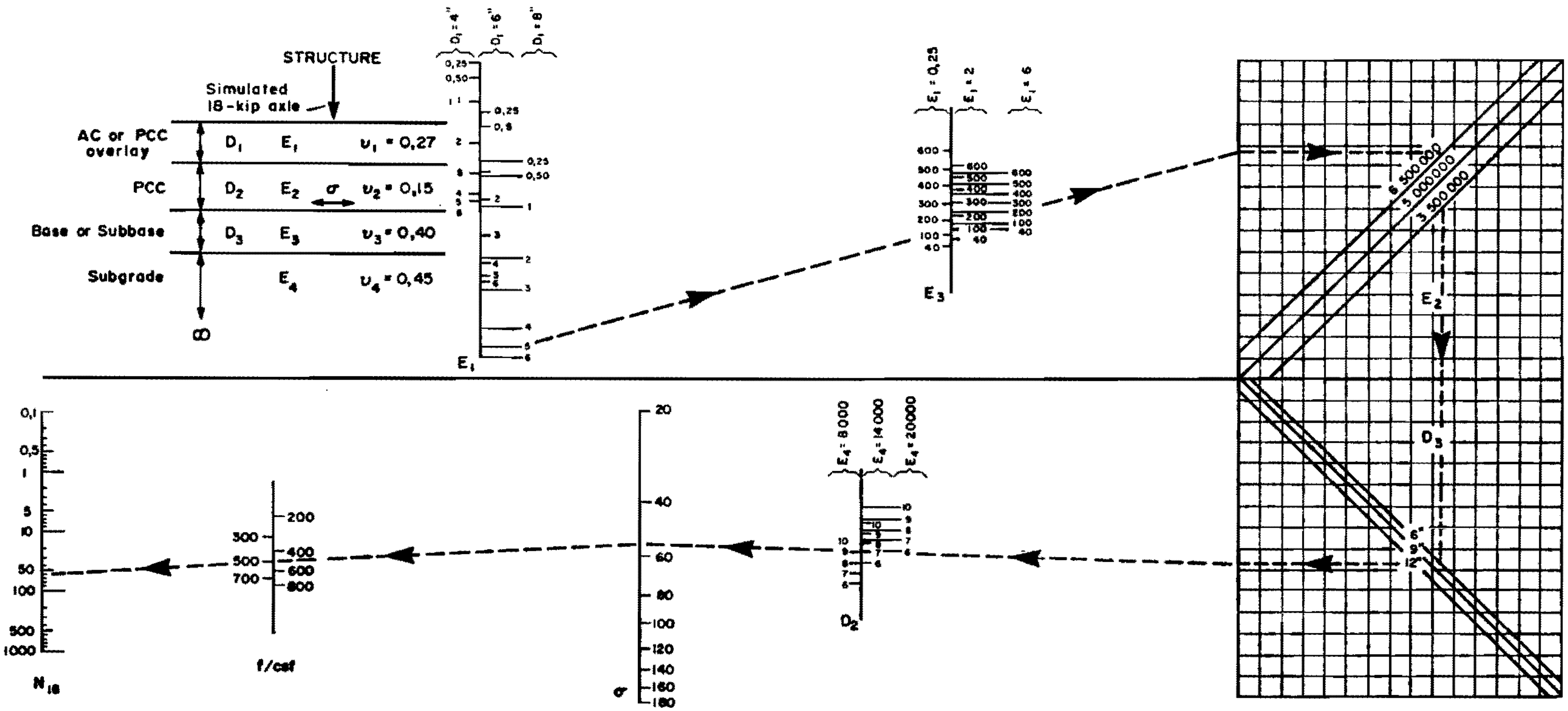
(c) ANSWER : D = 8,5" ≈ 9" (REFER TO BROKEN LINE)

Fig A12.4. Chart to determine the required thickness of an AC overlay.



WHERE :	APPLICATION
N_{18} = NUMBER OF 18-KIP ESAL APPLICATIONS TO TERMINAL DISTRESS CONDITION (10^6 18-KIP ESAL)	DETERMINE THE REQUIRED THICKNESS OF AN AC-OVERLAY
E_1 = MODULUS OF AC-OVERLAY (10^3 psi)	EXAMPLE
E_2 = MODULUS OF COMPOSITE LAYER (10^3 psi)	(a) GIVEN : E_1 = 800000 psi ; E_2 = 20000 psi ; REMAINING LIFE OF AC-OVERLAY = 100 %
D_1 = AC-OVERLAY THICKNESS (INCHES)	(b) REQUIRED : THICKNESS OF AC-OVERLAY TO CARRY 5×10^6 18-KIP ESAL
ϵ = TENSILE STRAIN (IN/IN x 10^{-3})	(c) ANSWER : D_1 = 8" (REFER TO BROKEN LINE)

Fig A12.5. Chart to determine the required thickness of an AC overlay.



WHERE :

N = NUMBER OF 18-KIP ESAL APPLICATIONS TO TERMINAL DISTRESS CONDITION OF EXISTING PCC-LAYER (10^6 18-KIP ESAL)

f = FLEXURAL STRENGTH OF EXISTING PCC LAYER (psi)

csf = CRITICAL STRESS FACTOR IN EXISTING PCC LAYER

E_1 = OVERLAY MODULUS (10^6 psi)

E_2 = MODULUS OF EXISTING PCC-LAYER (psi)

E_3 = SUBBASE MODULUS (10^3 psi)

E_4 = SUBGRADE MODULUS (psi)

D_i = LAYER THICKNESSES (IN INCHES)

APPLICATION

DETERMINE THE LIFE (IN 18-KIP ESAL) OF THE EXISTING PCC-LAYER AFTER OVERLAY PLACEMENT

EXAMPLE

(a) GIVEN : E_1 = 8 x 10^6 psi ; E_2 = 6,5 x 10^6 ; E_3 = 40 x 10^3 psi ; E_4 = 8000 psi ; D_1 = 6" ; D_2 = 8" ; D_3 = 6" ; f = 600 psi ; csf = 1,2 ; REMAINING LIFE OF EXISTING PCC-LAYER AT TIME OF OVERLAY PLACEMENT = 20 %

(b) REQUIRED : LIFE OF EXISTING PCC-LAYER AFTER OVERLAY PLACEMENT

(c) ANSWER : N_{18} = 63 x 10^6 (FROM CHART - REFER TO BROKEN LINE)
∴ LIFE OF EXISTING PCC-LAYER AFTER OVERLAY PLACEMEN = 63 x 10^6 x 20/100 ≈ 12,6 x 10^6 18-KIP ESAL

Fig A12.6. Chart to determine the life of the existing PCC layer after overlay placement.

ESAL's;

and $N_{18} = n_2 * 100/RL_o = 23.44 * 10^6 * 100/20 = 117.2 * 10^6$ 18k ESAL's

From the design chart in Fig A12.3, it follows that $D_1' = 13.5$ inches. The required overlay thickness = $D_1' - D_1$ = 13.5 inches - 8 inches = 5.5 inches. This overlay must be placed when the existing pavement reaches 20 percent remaining life. Theoretically, this will be 1.56 years from now if the cumulative equivalent traffic to time of overlay $(n_1) = 1.46 * 10^6$ 18k ESAL's and the traffic growth rate (i) of 3 percent is used in equation A4.2.

SECTION A.14: COST ANALYSIS

Strategy A

Cost Information. The following cost information is available:

(1) Real Discount Rate: r = 10 percent

(2) Construction Cost: C = \$15 per square yard ™ 0 years from now

(3) Maintenance Cost: M_1 = \$1 per square yard ™ 10 years from now
M_2 = \$1 per square yard ™ 15 years from now

(4) Salvage Value: S = \$5 per square yard ™ 20 years from now

(5) User Delay Cost: Not available

Cost Calculations (Refers to Table A13.1)

From Table A13.1 and the cost information presented above, it follows that

$$x_1 = 0 \text{ years};\ y_1 = 10 \text{ years};\ y_2 = 15 \text{ years};\ z = 20 \text{ years}$$

TABLE A13.1 DISCOUNT FACTOR (P) FOR CALCULATION OF NET PRESENT VALUE

Number of years (x, y or z) from the present to the time of money layout	* P for real discount rate r (r.p.a.)								
	2	3	4	5	6	7	8	9	10
1	1.02	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10
2	1.04	1.06	1.08	1.10	1.12	1.14	1.17	1.19	1.21
3	1.06	1.09	1.12	1.16	1.19	1.23	1.26	1.30	1.33
4	1.08	1.13	1.17	1.22	1.26	1.31	1.36	1.41	1.46
5	1.10	1.16	1.22	1.28	1.34	1.40	1.47	1.54	1.64
6	1.13	1.19	1.27	1.34	1.42	1.50	1.59	1.68	2.77
7	1.15	1.23	1.32	1.41	1.50	1.61	1.71	1.83	1.95
8	1.17	1.27	1.37	1.48	1.59	1.72	1.85	1.99	2.14
9	1.20	1.30	1.42	1.55	1.69	1.84	2.00	2.17	2.36
10	1.22	1.34	1.48	1.63	1.79	1.97	2.16	2.37	2.59
11	1.24	1.38	1.54	1.71	1.90	2.10	2.33	2.58	2.85
12	1.27	1.43	1.60	1.80	2.01	2.25	2.52	2.81	3.14
13	1.29	1.47	1.67	1.89	2.13	2.41	2.72	3.07	3.45
14	1.32	1.51	1.73	1.98	2.26	2.58	2.94	3.34	3.80
15	1.35	1.56	1.80	2.08	2.40	2.76	3.17	3.64	4.18
16	1.37	1.60	1.87	2.18	2.54	2.95	3.43	3.97	4.59
17	1.40	1.65	1.95	2.29	2.69	3.16	3.70	4.33	5.05
18	1.43	1.70	2.03	2.41	2.85	3.38	4.00	4.72	5.56
19	1.46	1.75	2.11	2.53	3.03	3.62	4.32	5.14	6.12
20	1.49	1.81	2.19	2.65	3.21	3.87	4.66	5.60	6.73
21	1.52	1.86	2.28	2.79	3.40	4.14	5.03	6.11	7.40
22	1.55	1.92	2.37	2.93	3.60	4.43	5.44	6.66	8.14
23	1.58	1.97	2.46	3.07	3.82	4.74	5.87	7.26	8.95
24	1.61	2.03	2.56	3.23	4.05	5.07	6.34	7.92	9.85
25	1.64	2.09	2.67	3.39	4.29	5.43	6.85	8.62	10.83
26	1.67	2.16	2.77	3.56	4.55	5.81	7.40	9.40	11.92
27	1.71	2.22	2.88	3.73	4.82	6.21	7.99	10.25	13.11
28	1.74	2.29	3.00	3.92	5.11	6.65	8.63	11.17	14.42
29	1.78	2.36	3.12	4.12	5.42	7.11	9.32	12.17	15.86
30	1.81	2.43	3.24	4.32	5.74	7.61	10.06	13.27	17.45

*$P = (1 + 0.01i)^{x, y \text{ or } z}$

Net Present Value of construction and maintenance cost =

$$\frac{C}{P_{x_1}} + \frac{M_1}{P_{y_i}} + \frac{M_2}{P_{y_2}} - \frac{S}{P_z}$$

$$= \frac{15}{P_o} + \frac{1}{P_{10}} + \frac{1}{P_{15}} - \frac{5}{P_{20}}$$

$$= \frac{15}{1} + \frac{1}{2.59} + \frac{1}{4.18} - \frac{5}{6.73} \quad \ldots \text{A13.1}$$

$= \$14.88$ per square yard.

From Table A10.1(a) the user delay cost is approximately 71 percent of the total net present value.

$$\therefore \text{TNPV} = 14.88 + 0.71 * (\text{total net present value})$$

$$\therefore \text{TNPV} = \frac{14.88}{0.29} = \$51.32 \text{ per square yard}$$

$$= \$50.00 \text{ per square yard}$$

<u>Strategy B</u>

<u>Cost Information</u>. The following information is available:

(1) Real Discount Rate: $r = 10$ percent

(2) Construction Cost: $C = \$17$ per square yard ™ 1.5 years from now

(3) Maintenance Cost: $M_1 = \$1$ per square yard ™ 10 years from now

$M_2 = \$2$ per square yard ™ 15 years from now

(4) Salvage Value: $S = \$$ per square yard ™ 20 years from now

<u>Cost Calculations (Refers to Table A13.1)</u>

From Table A13.1 and the cost information presented above, it follows that

$x_1 = 2$ years; $y_1 = 10$ years; $y_2 = 15$ years; $z = 20$ years

$$\therefore \text{NPV} = \frac{C}{P_{x_1}} + \frac{M}{P_{y_1}} + \frac{M_2}{P_{y_2}} - \frac{S}{P_z}$$

$$= \frac{17}{P_2} + \frac{1}{P_{10}} + \frac{2}{P_{15}} - \frac{5}{P_{20}}$$

$$= \frac{17}{1.15} + \frac{1}{2.59} + \frac{2}{4.18} - \frac{5}{6.73}$$

$$= \$14.77$$

From Table A10.1(a) the user delay cost is approximately 82 percent of the STNPV.

$$\text{TNPV} = 14.77 + 0.82\ (\text{TNPV})$$

$$\text{TNPV} = \$82.09$$

$$\$80.00$$

Cost Comparison

The cost ratio = $\frac{\$50\ (\text{Strategy A})}{\$80\ (\text{Strategy B})} = 1.6$

This value compares well with estimated cost ratio of 1.55 predicted in Table A10.1(a).

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A1. Schnitter, Otto, W. R. Hudson, and B. F. McCullough, "A Rigid Pavement Overlay Design Procedure for the Texas SDHPT," Research Report 177-13, Center for Highway Research, The University of Texas at Austin, May 1978.

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A3. Uddin, W., V. Torres-Verdin, W. R. Hudson, A. H. Meyer and B. F. McCullough, "A User Manual of Dynaflect Testing for Rigid Pavement," Research Report 256-6F, Center for Transportation Research, The University of Texas at Austin, August 1983.

A4. "ELSYM5 3/72-3, Elastic Layered System with One to Ten Normal Identical Circular Uniform Loads," Unpublished computer application, by Gale Ahlborn, Institute of Transportation and Traffic Engineering, University of California at Berkeley, 1972.

A5. Seeds, S. B., W. R. Hudson, and B. F. McCullough, "A Design System for Rigid Pavement Rehabilitation," Research Report 249-2, Center for Transportation Research, The University of Texas at Austin, 1980.

A6. Seeds, S. B., B. F. McCullough, and R. F. Carmichael, "Arkansas Reflection Cracking Analysis and Overlay Design Procedure," Final Report No. UA-3/1, 1983.

A7. Taute, A. and B. F. McCullough, "Manual for Condition Survey of CRCP," Research Report 177-19, Center for Transportation Research, The University of Texas at Austin, 1979.

A8. Gutierrez de Velasco, M. and B. F. McCullough, "Rigid Pavement Network Rehabilitation Scheduling Using Distress Quantities," Research Report 249-5, Center for Transportation Research, The University of Texas at Austin.

A9. "Operations and Procedures Manual," SDHPT, Highway Design Division.

A10. Anagnos, James N. and Thomas W. Kennedy, "Practical Method of

Conducting the Indirect Tensile Test," Research Report 98-10, Center for Highway Research, The University of Texas at Austin, August 1972.

A11. Mendoza-Diaz, Alberto and B. F. McCullough, "Design Charts for the Design of ACHM Overlays on PCC Pavements Against Reflection Cracking," Research Report 249-6, Center for Transportation Research, The University of Texas at Austin, August 1983.

A12. Torres, Francisco and B. F. McCullough, "Void Detection and Grouting Process," Research Report 249-3, Center for Transportation Research, The University of Texas at Austin, May 1982.

APPENDIX B. INPUTS TO RPRDS-1 FOR DEVELOPING DESIGN STRATEGY TABLES

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 CCCCCCCCCC      RRRRRRRRRRR       CCCCCCCCCC      PPPPPPPPPPP
CCCCCCCCCCCC     RRRRRRRRRRRR     CCCCCCCCCCCC     PPPPPPPPPPPP
CC        CC     RR        RR     CC        CC     PP        PP
CC               RR        RR     CC               PP        PP
CC               RR        RR     CC               PP        PP
CC               RR        RR     CC               PP        PP
CC               RR        RR     CC               PPPPPPPPPPPP
CC               RRRRRRRRRRRR     CC               PPPPPPPPPPP
CC               RRRRRRRRRRR      CC               PP
CC               RR    RR         CC               PP
CC               RR     RR        CC               PP
CC               RR      RR       CC               PP
CC               RR       RR      CC               PP
CC        CC     RR        RR     CC        CC     PP
CCCCCCCCCCCC     RR        RR     CCCCCCCCCCCC     PP
 CCCCCCCCCC      RR        RR      CCCCCCCCCC      PP
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PRDS1 - PAVEMENT REHABILITATION DESIGN SYSTEM - VERSION 1, APRIL 1982
CENTER FOR TRANSPORTATION RESEARCH
UNIVERSITY OF TEXAS AT AUSTIN
LATEST REVISION - ARE INC., CONSULTING ENGINEERS, JULY 1982

****
P R D S   I N P U T   S U M M A R Y
****

PROJECT DESCRIPTION
****

1.1 TITLE
A W VILJOEN CRCP

ORIGINAL PAVEMENT
****

2.1	SURFACE TYPE		CRCP
2.2	CONCRETE SHOULDER		NO
2.3	NO. OF LANES (ONE DIRECTION)		2
2.4	NO. OF PAVEMENT LAYERS		3
3.1	PROJECT LENGTH, MILES		1.00
3.2	LANE WIDTH, FEET		12.0
3.3	TOTAL SHOULDER WIDTH, FEET		14.

PAVEMENT STRUCTURE

LAYER NO.	4.0 THICKNESS (IN)	5.0 ELASTIC MODULUS (PSI)	6.0 POISSONS RATIO	
1	8.0	5000000.	.20	
2	8.0	200000.	.30	
3	SEMI-INFINITE	14000.	.45	*

7.1	CONCRETE FLEXURAL STRENGTH, PSI		690.
7.2	CRITICAL STRESS FACTOR		1.20
7.3	CONCRETE STIFFNESS AFTER CRACKING, PSI		800000.
8.1	NO. OF EXISTING DEFECTS PER MILE	**	15.
8.2	COST OF REPAIRING A DEFECT, DOL		1000.
8.3	RATE OF DEFECT DEVELOPMENT, NO./YR/MILE	***	4.

* Layer No. 3 modulus range from 8,000 to 20,000 in steps of 6,000

** Range from 4 to 15 depending on RL of existing pavement

*** Range from 2 to 4 depending on RL of existing pavement

TRAFFIC VARIABLES
****

9.1	AVERAGE DAILY TRAFFIC (ADT)	*	60000.
9.2	ADT GROWTH RATE, PERCENT		4.00
9.3	INITIAL YEARLY 18-KIP ESAL, MILLIONS	**	2.190
9.4	18-KIP ESAL GROWTH RATE, PERCENT		4.00
9.5	DIRECTIONAL DISTRIBUTION FACTOR, PERCENT		50.0
9.6	LANE DISTRIBUTION FACTOR, PERCENT		90.0

TIME CONSTRAINTS
****

10.1	ANALYSIS PERIOD, YEARS	20.0
10.2	MINIMUM TIME BETWEEN OVERLAYS, YEARS	10.0
10.3	MAXIMUM ALLOWABLE YEARS OF HEAVY MAINTENANCE AFTER LOSS OF STRUCTURAL LOAD-CARRYING CAPACITY	4.0

* Separate runs for ADT of 15,000 and 30,000

** Separate runs for initial yearly ESAL of 1.100 and 0.567

REMAINING LIFE VARIABLES
****

11.1 NO. OF ORIGINAL PAVEMENT REMAINING LIFE VALUES TO CONSIDER * 1

11.2 MINIMUM EXISTING PAVEMENT REMAINING LIFE BELOW WHICH A BONDED PCC OVERLAY MAY NOT BE PLACED 10.

11.3 VALUES OF ORIGINAL PAVEMENT REMAINING LIFE AT WHICH FIRST OVERLAY MAY BE PLACED

NO.	REMAINING LIFE (PERCENT)	
1	0	**

12.1 NO. OF FIRST OVERLAY REMAINING LIFE VALUES TO CONSIDER 4

12.2 VALUES OF FIRST OVERLAY REMAINING LIFE AT WHICH SECOND OVERLAY MAY BE PLACED

NO.	REMAINING LIFE (PERCENT)
1	40.
2	20.
3	10.
4	0

* Depends on RL of existing pavement (RL):

4 when RL = 40 percent
3 when RL = 20 percent
2 when RL = 10 percent
1 when RL = 0 percent

** Depends on RL:

40; 20; 10 and 0	when RL = 40 percent
20; 10 and 0	when RL = 20 percent
10 and 0	when RL = 10 percent
0	when RL = 0 percent

```
OVERLAY CHARACTERISTICS
***********************

  13.0   TYPES OF FIRST OVERLAY TO CONSIDER
           .1   ACP               = YES
           .2   BONDED CRCP       = YES
           .3   UNBONDED CRCP     = YES
           .4   BONDED JCP        = YES
           .5   UNBONDED JCP      = YES
  14.0   TYPES OF SECOND OVERLAY TO CONSIDER
           .1   ACP  = YES
           .2   CRCP =  NO
           .3   JCP  =  NO

  15.0   NO. OF DIFFERENT OVERLAY THICKNESS TO CONSIDER
           .1   ACP FIRST OVERLAY   =    7
           .2   ACP SECOND OVERLAY  =    3
           .3   PCC OVERLAY         =    7

  16.0   ACP FIRST OVERLAY THICKNESSES, INCHES
           .1     2.0
           .2     3.0
           .3     4.0
           .4     5.0
           .5     6.0
           .6     7.0
           .7     8.0

  17.0   ACP SECOND OVERLAY THICKNESSES, INCHES
           .1     2.0
           .2     4.0
           .3     6.0
  18.0   PCC OVERLAY THICKNESSES, INCHES
           .1     3.0
           .2     4.0
           .3     5.0
           .4     6.0
           .5     7.0
           .6     8.0
           .7     9.0
```

19.1	ALLOWABLE TOTAL OVERLAY THICKNESS, INCHES	25.0
19.2	AVERAGE LEVEL-UP THICKNESS, INCHES	1.0
19.3	BOND BREAKER THICKNESS, INCHES	1.0
20.1	ACP OVERLAY DESIGN STIFFNESS, PSI	500000.
20.2	POISSONS RATIO, ACP OVERLAY	.30
20.3	PCC OVERLAY DESIGN STIFFNESS, PSI	5000000.
20.4	POISSONS RATIO, PCC OVERLAY	.20
20.5	BOND BREAKER STIFFNESS, PSI	100000.
20.6	POISSONS RATIO, BOND BREAKER	.35
21.1	NO. OF OVERLAY FLEXURAL STRENGTHS TO CONSIDER	1
21.2	NO. WHICH IDENTIFIES WHICH FLEXURAL STRENGTH IN THE LIST TO USE FOR A BONDED PCC OVERLAY	1

22.0 PCC OVERLAY FLEXURAL STRENGTH(S), PSI
.1 650.

*** PAVEMENT STRESS FACTORS AFTER OVERLAY ***

	FIRST OVERLAY TYPE	SECOND OVERLAY TYPE	CRITICAL STRESS LOCATION	OVERLAY SHOULDER TYPE	CRIT./INTER. STRESS FACTOR
23.1	ACP	(NONE)	EX PAVT	ACP	1.25
24.1	ACP	ACP	EX PAVT	ACP	1.25
25.1	ACP	CRCP	EX PAVT	ACP	.0
25.2	ACP	CRCP	EX PAVT	CRCP	.0
26.1	ACP	CRCP	CRCP O/L	ACP	.0
26.2	ACP	CRCP	CRCP O/L	CRCP	.0
27.1	ACP	JCP	EX PAVT	ACP	.0
27.2	ACP	JCP	EX PAVT	JCP	.0
28.1	ACP	JCP	JCP O/L	ACP	.0
28.2	ACP	JCP	JCP O/L	JCP	.0
29.1	BOND CRC	(NONE)	EX PAVT	ACP	1.25
29.2	BOND CRC	(NONE)	EX PAVT	CRCP	1.08
30.1	BOND CRC	ACP	EX PAVT	ACP	1.25
30.2	BOND CRC	ACP	EX PAVT	CRCP	1.08
31.1	BOND JCP	(NONE)	EX PAVT	ACP	.0
31.2	BOND JCP	(NONE)	EX PAVT	JCP	.0
32.1	BOND JCP	ACP	EX PAVT	ACP	.0
32.2	BOND JCP	ACP	EX PAVT	JCP	.0
33.1	UNBD CRC	(NONE)	EX PAVT	ACP	1.25
33.2	UNBD CRC	(NONE)	EX PAVT	CRCP	1.08
34.1	UNBD CRC	(NONE)	CRCP O/L	ACP	1.25
34.2	UNBD CRC	(NONE)	CRCP O/L	CRCP	1.08
35.1	UNBD CRC	ACP	EX PAVT	ACP	1.25
35.2	UNBD CRC	ACP	EX PAVT	CRCP	1.08
36.1	UNBD CRC	ACP	CRCP O/L	ACP	1.25
36.2	UNBD CRC	ACP	CRCP O/L	CRCP	1.08
37.1	UNBD JCP	(NONE)	EX PAVT	ACP	1.40
37.2	UNBD JCP	(NONE)	EX PAVT	JCP	1.20
38.1	UNBD JCP	(NONE)	JCP O/L	ACP	1.40
38.2	UNBD JCP	(NONE)	JCP O/L	JCP	1.15
39.1	UNBD JCP	ACP	EX PAVT	ACP	1.40
39.2	UNBD JCP	ACP	EX PAVT	JCP	1.20
40.1	UNBD JCP	ACP	JCP O/L	ACP	1.40
40.2	UNBD JCP	ACP	JCP O/L	JCP	1.15

NOTE - STRATEGIES WITH A ZERO VALUE FOR THE CRITICAL TO INTERIOR STRESS FACTOR WILL NOT BE CONSIDERED.

41.1 2 - REGRESSION EQUATIONS USED TO PREDICT RESPONSE.

OVERLAY CONSTRUCTION COST VARIABLES
****

42.0 SITE ESTABLISHMENT COST, DOL

.1	ACP EQUIPMENT	10000.
.2	CRCP EQUIPMENT	20000.
.3	JCP EQUIPMENT	20000.
.4	ACP AND CRCP EQUIPMENT	25000.
.5	ACP AND JCP EQUIPMENT	25000.

43.0 PAVEMENT SURFACE PREPARATION COSTS, DOL/SY

.1	EXISTING PAVEMENT	1.20
.2	ACP OVERLAY	.20
.3	CRCP OVERLAY	.30
.4	JCP OVERLAY	.30

44.1	FIXED COST OF ACP OVERLAY CONSTRUCTION, DOL/SY	0
44.2	VARIABLE COST OF ACP OVERLAY CONSTR., DOL/SY/IN	1.72
44.3	FIXED COST OF FLEXIBLE SHOULDER CONSTR., DOL/SY	7.75
44.4	VARIABLE COST OF FLEX. SHOULDER CONSTR., DOL/SY/IN	0
44.5	COST OF BOND BREAKER CONSTRUCTION, DOL/SY	3.44

45.0 CRCP FIXED COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	FIXED COST (DOL/SY)
.1	650.	6.00

46.0 CRCP VARIABLE COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	VARIABLE COST (DOL/SY/IN)
.1	650.	1.00

47.0 JCP FIXED COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	FIXED COST (DOL/SY)
	-------------	---------
1	650.	7.00

48.0 JCP VARIABLE COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	VARIABLE COST (DOL/SY/IN)
	-------------	-------------
1	650.	1.80

49.1 TOTAL STEEL PERCENTAGE REQUIRED IN CRCP OVERLAYS .0
49.2 TOTAL STEEL PERCENTAGE REQUIRED IN JCP OVERLAYS .0
49.3 COST OF STEEL REINFORCEMENT, DOL/LB .0

TRAFFIC DELAY COST VARIABLES
****

50.1	LOCATION OF PROJECT (1=RURAL,2=URBAN)	1
50.2	MODEL NO. FOR HANDLING TRAFFIC	3
50.3	NO. OF OPEN LANES, OVERLAY DIRECTION	1
50.4	NO. OF OPEN LANES, NON-OVERLAY DIRECTION	2
51.1	MILITARY TIME OVERLAY CONSTRUCTION BEGINS	-0
51.2	MILITARY TIME OVERLAY CONSTRUCTION ENDS	2400.
51.3	HOURS PER DAY OVERLAY CONSTRUCTION OCCURS	10.0
51.4	NO. OF DAYS CONCRETE IS ALLOWED TO CURE	14.
51.5	DETOUR DISTANCE TO USE IN MODEL 5, MILES	0
52.1	AVERAGE APPROACH SPEED, MPH	55.
52.2	AVERAGE SPEED, OVERLAY DIRECTION, MPH	30.
52.3	AVERAGE SPEED, NON-OVERLAY DIRECTION, MPH	55.
53.1	DISTANCE TRAFFIC IS SLOWED, OVERLAY DIRECTION, MILES	1.0
53.2	DISTANCE TRAFFIC IS SLOWED, NON-OVERLAY DIR., MILES	0
53.3	PERCENT OF VEHICLES STOPPED, OVERLAY DIRECTION	0
53.4	PERCENT OF VEHICLES STOPPED, NON-OVERLAY DIRECTION	0
53.5	AVERAGE VEHICLE DELAY, OVERLAY DIRECTION, HRS	-0
53.6	AVERAGE VEHICLE DELAY, NON-OVERLAY DIRECTION, HRS	-0
54.1	ACP PRODUCTION RATE, CY/HR	70.
54.2	CRCP PRODUCTION RATE, CY/HR	70.
54.3	JCP PRODUCTION RATE, CY/HR	70.
54.4	BOND BREAKER PRODUCTION RATE, CY/HR	70.

DISTRESS/MAINTENANCE COST VARIABLES
****

55.1 DISTRESS REPAIR COST, CRCP OVERLAY, DOL 1000.00
55.2 INITIAL CRCP OVERLAY DISTRESS RATE, NO./MI/YR 1.0
55.3 SECONDARY CRCP OVERLAY DISTRESS RATE, NO./MI/YR 2.0
55.4 CRCP OVERLAY DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD-CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	3.0
2	5.0
3	8.0
4	10.0

56.1 DISTRESS REPAIR COST, JCP OVERLAY, DOL 600.00
56.2 INITIAL JCP OVERLAY DISTRESS RATE, NO./MI/YR 1.0
56.3 SECONDARY JCP OVERLAY DISTRESS RATE, NO./MI/YR 2.0
56.4 JCP OVERLAY DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	4.0
2	6.0
3	9.0
4	10.0

57.1 DISTRESS REPAIR COST, ACP OVERLAY ON CRCP, DOL 500.00
57.2 INITIAL ACP/CRCP DISTRESS RATE, NO./MI/YR 1.2
57.3 SECONDARY ACP/CRCP DISTRESS RATE, NO./MI/YR 2.4
57.4 ACP/CRCP DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD-CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	3.6
2	6.0
3	9.0
4	18.0

58.1 DISTRESS REPAIR COST, ACP OVERLAY ON JCP, DOL 500.00
58.2 INITIAL ACP/JCP DISTRESS RATE, NO./MI/YR 2.0
58.3 SECONDARY ACP/JCP DISTRESS RATE, NO./MI/YR 4.0
58.4 ACP/JCP DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	4.0
2	7.0
3	10.0
4	20.0

59.1	DISTRESS REPAIR COST, ACP OVERLAY ON ACP, DOL	-0
59.2	INITIAL ACP/ACP DISTRESS RATE, NO./MI/YR	-0
59.3	SECONDARY ACP/ACP DISTRESS RATE, NO./MI/YR	-0
59.4	ACP/ACP DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD CARRYING CAPACITY	

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	-0
2	-0
3	-0
4	-0

COST RETURNS
****

60.1	SALVAGE VALUE, PERCENT OF OVERLAY CONSTRUCTION COST	10.0
60.2	VALUE OF EACH YEAR OF EXTENDED LIFE, DOL/SY/YR	0

COMBINED INTEREST AND INFLATION RATE
****

61.1	INTEREST RATE MINUS INFLATION RATE, PERCENT	5.0

```
          JJ     CCCCCCCCCC     PPPPPPPPPPP
          JJ    CCCCCCCCCCCC    PPPPPPPPPPPP
          JJ    CC        CC    PP        PP
          JJ    CC              PP        PP
          JJ    CC              PP        PP
          JJ    CC              PP        PP
          JJ    CC              PPPPPPPPPPPP
          JJ    CC              PPPPPPPPPPP
          JJ    CC              PP
          JJ    CC              PP
          JJ    CC              PP
          JJ    CC              PP
JJ        JJ    CC              PP
JJ        JJ    CC        CC    PP
JJJJJJJJJJJJ    CCCCCCCCCCCC    PP
 JJJJJJJJJJ      CCCCCCCCCC     PP
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PRDS1 - PAVEMENT REHABILITATION DESIGN SYSTEM - VERSION 1, APRIL 1982
CENTER FOR TRANSPORTATION RESEARCH
UNIVERSITY OF TEXAS AT AUSTIN
LATEST REVISION - ARE INC., CONSULTING ENGINEERS, JULY 1982


*************************************
P R D S   I N P U T   S U M M A R Y
*************************************


  PROJECT DESCRIPTION
  *******************

     1.1  TITLE
  VILJOEN    JCP RUN1


  ORIGINAL PAVEMENT
  *****************

     2.1  SURFACE TYPE                                                    JCP
     2.2  CONCRETE SHOULDER                                                NO
     2.3  NO. OF LANES (ONE DIRECTION)                                      2
     2.4  NO. OF PAVEMENT LAYERS                                            3

     3.1  PROJECT LENGTH, MILES                                          1.00
     3.2  LANE WIDTH, FEET                                               12.0
     3.3  TOTAL SHOULDER WIDTH, FEET                                      14.

          PAVEMENT STRUCTURE
                                     5.0
                     4.0           ELASTIC      6.0
             LAYER   THICKNESS     MODULUS    POISSONS
              NO.      (IN)         (PSI)      RATIO
             ----- ------------- --------- --------
               1       10.0       5000000.      .20
               2       10.0        200000.      .30
               3   SEMI-INFINITE   14000.       .45          *


     7.1  CONCRETE FLEXURAL STRENGTH, PSI                                 690.
     7.2  CRITICAL STRESS FACTOR                                          1.35
     7.3  CONCRETE STIFFNESS AFTER CRACKING, PSI                       400000.

     8.1  NO. OF EXISTING DEFECTS PER MILE                   **            16.
     8.2  COST OF REPAIRING A DEFECT, DOL                                1000.
     8.3  RATE OF DEFECT DEVELOPMENT, NO./YR/MILE            ***            4.
```

* Layer 3 modulus range from 8,000 to 20,000 in steps of 6,000

** Range from 4 to 16 depending on RL of existing pavement

*** Range from 2 to 4 depending on RL of existing pavement

TRAFFIC VARIABLES
****

9.1	AVERAGE DAILY TRAFFIC (ADT)	*	60000.
9.2	ADT GROWTH RATE, PERCENT		4.00
9.3	INITIAL YEARLY 18-KIP ESAL, MILLIONS	**	2.190
9.4	18-KIP ESAL GROWTH RATE, PERCENT		4.00
9.5	DIRECTIONAL DISTRIBUTION FACTOR, PERCENT		50.0
9.6	LANE DISTRIBUTION FACTOR, PERCENT		90.0

TIME CONSTRAINTS
****

10.1	ANALYSIS PERIOD, YEARS	20.0
10.2	MINIMUM TIME BETWEEN OVERLAYS, YEARS	10.0
10.3	MAXIMUM ALLOWABLE YEARS OF HEAVY MAINTENANCE AFTER LOSS OF STRUCTURAL LOAD-CARRYING CAPACITY	4.0

* Separate rung for ADT of 15,000 and 30,000

** Separate rung for initial yearly ESAL of 1.100 and 0.567

REMAINING LIFE VARIABLES
****

11.1 NO. OF ORIGINAL PAVEMENT REMAINING LIFE VALUES TO CONSIDER * 1

11.2 MINIMUM EXISTING PAVEMENT REMAINING LIFE BELOW WHICH A BONDED PCC OVERLAY MAY NOT BE PLACED 10.

11.3 VALUES OF ORIGINAL PAVEMENT REMAINING LIFE AT WHICH FIRST OVERLAY MAY BE PLACED

NO.	REMAINING LIFE (PERCENT)
1	0 **

12.1 NO. OF FIRST OVERLAY REMAINING LIFE VALUES TO CONSIDER 4

12.2 VALUES OF FIRST OVERLAY REMAINING LIFE AT WHICH SECOND OVERLAY MAY BE PLACED

NO.	REMAINING LIFE (PERCENT)
1	40.
2	20.
3	10.
4	0

* Depends on RL of existing pavement (RL):

4 when RL = 40 percent
3 when RL = 20 percent
2 when RL = 10 percent
1 when RL = 0 percent

** Depends on RL:

40; 20; 10 and 0	when RL = 40 percent
20; 10 and 0	when RL = 20 percent
10 and 0	when RL = 10 percent
0	when RL = 0 percent

```
OVERLAY CHARACTERISTICS
************************

 13.0  TYPES OF FIRST OVERLAY TO CONSIDER
       .1  ACP              = YES
       .2  BONDED CRCP      =  NO
       .3  UNBONDED CRCP    = YES
       .4  BONDED JCP       = YES
       .5  UNBONDED JCP     = YES

 14.0  TYPES OF SECOND OVERLAY TO CONSIDER
       .1  ACP  = YES
       .2  CRCP =  NO
       .3  JCP  =  NO

 15.0  NO. OF DIFFERENT OVERLAY THICKNESS TO CONSIDER
       .1  ACP FIRST OVERLAY  =    6
       .2  ACP SECOND OVERLAY =    3
       .3  PCC OVERLAY        =    7

 16.0  ACP FIRST OVERLAY THICKNESSES, INCHES
       .1     3.0
       .2     4.0
       .3     5.0
       .4     6.0
       .5     7.0
       .6     8.0

 17.0  ACP SECOND OVERLAY THICKNESSES, INCHES
       .1     2.0
       .2     4.0
       .3     6.0

 18.0  PCC OVERLAY THICKNESSES, INCHES
       .1     3.0
       .2     4.0
       .3     5.0
       .4     6.0
       .5     7.0
       .6     8.0
       .7     9.0
```

19.1	ALLOWABLE TOTAL OVERLAY THICKNESS, INCHES	25.0
19.2	AVERAGE LEVEL-UP THICKNESS, INCHES	1.5
19.3	BOND BREAKER THICKNESS, INCHES	1.5
20.1	ACP OVERLAY DESIGN STIFFNESS, PSI	500000.
20.2	POISSONS RATIO, ACP OVERLAY	.30
20.3	PCC OVERLAY DESIGN STIFFNESS, PSI	5000000.
20.4	POISSONS RATIO, PCC OVERLAY	.20
20.5	BOND BREAKER STIFFNESS, PSI	100000.
20.6	POISSONS RATIO, BOND BREAKER	.35
21.1	NO. OF OVERLAY FLEXURAL STRENGTHS TO CONSIDER	1
21.2	NO. WHICH IDENTIFIES WHICH FLEXURAL STRENGTH IN THE LIST TO USE FOR A BONDED PCC OVERLAY	1
22.0	PCC OVERLAY FLEXURAL STRENGTH(S), PSI	
	.1 650.	

*** PAVEMENT STRESS FACTORS AFTER OVERLAY ***

	FIRST OVERLAY TYPE	SECOND OVERLAY TYPE	CRITICAL STRESS LOCATION	OVERLAY SHOULDER TYPE	CRIT./INTER. STRESS FACTOR
23.1	ACP	(NONE)	EX PAVT	ACP	1.45
24.1	ACP	ACP	EX PAVT	ACP	1.45
25.1	ACP	CRCP	EX PAVT	ACP	-0
25.2	ACP	CRCP	EX PAVT	CRCP	-0
26.1	ACP	CRCP	CRCP O/L	ACP	-0
26.2	ACP	CRCP	CRCP O/L	CRCP	-0
27.1	ACP	JCP	EX PAVT	ACP	-0
27.2	ACP	JCP	EX PAVT	JCP	-0
28.1	ACP	JCP	JCP O/L	ACP	-0
28.2	ACP	JCP	JCP O/L	JCP	-0
29.1	BOND CRC	(NONE)	EX PAVT	ACP	-0
29.2	BOND CRC	(NONE)	EX PAVT	CRCP	-0
30.1	BOND CRC	ACP	EX PAVT	ACP	-0
30.2	BOND CRC	ACP	EX PAVT	CRCP	-0
31.1	BOND JCP	(NONE)	EX PAVT	ACP	1.45
31.2	BOND JCP	(NONE)	EX PAVT	JCP	1.20
32.1	BOND JCP	ACP	EX PAVT	ACP	1.45
32.2	BOND JCP	ACP	EX PAVT	JCP	1.20
33.1	UNBD CRC	(NONE)	EX PAVT	ACP	1.35
33.2	UNBD CRC	(NONE)	EX PAVT	CRCP	1.20
34.1	UNBD CRC	(NONE)	CRCP O/L	ACP	1.30
34.2	UNBD CRC	(NONE)	CRCP O/L	CRCP	1.15
35.1	UNBD CRC	ACP	EX PAVT	ACP	1.30
35.2	UNBD CRC	ACP	EX PAVT	CRCP	1.15
36.1	UNBD CRC	ACP	CRCP O/L	ACP	1.25
36.2	UNBD CRC	ACP	CRCP O/L	CRCP	1.00
37.1	UNBD JCP	(NONE)	EX PAVT	ACP	1.40
37.2	UNBD JCP	(NONE)	EX PAVT	JCP	1.20
38.1	UNBD JCP	(NONE)	JCP O/L	ACP	1.40
38.2	UNBD JCP	(NONE)	JCP O/L	JCP	1.20
39.1	UNBD JCP	ACP	EX PAVT	ACP	1.40
39.2	UNBD JCP	ACP	EX PAVT	JCP	1.25
40.1	UNBD JCP	ACP	JCP O/L	ACP	1.40
40.2	UNBD JCP	ACP	JCP O/L	JCP	1.20

NOTE - STRATEGIES WITH A ZERO VALUE FOR THE CRITICAL TO INTERIOR STRESS FACTOR WILL NOT BE CONSIDERED.

41.1 2 - REGRESSION EQUATIONS USED TO PREDICT RESPONSE.

OVERLAY CONSTRUCTION COST VARIABLES
****

42.0 SITE ESTABLISHMENT COST, DOL

01	ACP EQUIPMENT	10000.
02	CRCP EQUIPMENT	20000.
03	JCP EQUIPMENT	20000.
04	ACP AND CRCP EQUIPMENT	25000.
05	ACP AND JCP EQUIPMENT	25000.

43.0 PAVEMENT SURFACE PREPARATION COSTS, DOL/SY

01	EXISTING PAVEMENT	1.20
02	ACP OVERLAY	.20
03	CRCP OVERLAY	.30
04	JCP OVERLAY	.30

44.1	FIXED COST OF ACP OVERLAY CONSTRUCTION, DOL/SY	1.50
44.2	VARIABLE COST OF ACP OVERLAY CONSTR., DOL/SY/IN	1.72
44.3	FIXED COST OF FLEXIBLE SHOULDER CONSTR., DOL/SY	7.75
44.4	VARIABLE COST OF FLEX. SHOULDER CONSTR., DOL/SY/IN	0
44.5	COST OF BOND BREAKER CONSTRUCTION, DOL/SY	5.16

45.0 CRCP FIXED COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	FIXED COST (DOL/SY)
01	650.	6.00

46.0 CRCP VARIABLE COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	VARIABLE COST (DOL/SY/IN)
01	650.	1.80

47.0 JCP FIXED COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	FIXED COST (DOL/SY)
	--------------	----------
01	650.	7.00

48.0 JCP VARIABLE COST FOR EACH FLEXURAL STRENGTH

	FLEXURAL STRENGTH (PSI)	VARIABLE COST (DOL/SY/IN)
	--------------	-------------
01	650.	1.00

49.1 TOTAL STEEL PERCENTAGE REQUIRED IN CRCP OVERLAYS =0
49.2 TOTAL STEEL PERCENTAGE REQUIRED IN JCP OVERLAYS =0
49.3 COST OF STEEL REINFORCEMENT, DOL/LB =0

TRAFFIC DELAY COST VARIABLES
****

50.1	LOCATION OF PROJECT (1=RURAL,2=URBAN)	1
50.2	MODEL NO. FOR HANDLING TRAFFIC	3
50.3	NO. OF OPEN LANES, OVERLAY DIRECTION	1
50.4	NO. OF OPEN LANES, NON-OVERLAY DIRECTION	2
51.1	MILITARY TIME OVERLAY CONSTRUCTION BEGINS	-0
51.2	MILITARY TIME OVERLAY CONSTRUCTION ENDS	2400.
51.3	HOURS PER DAY OVERLAY CONSTRUCTION OCCURS	10.0
51.4	NO. OF DAYS CONCRETE IS ALLOWED TO CURE	14.
51.5	DETOUR DISTANCE TO USE IN MODEL 5, MILES	0
52.1	AVERAGE APPROACH SPEED, MPH	55.
52.2	AVERAGE SPEED, OVERLAY DIRECTION, MPH	30.
52.3	AVERAGE SPEED, NON-OVERLAY DIRECTION, MPH	55.
53.1	DISTANCE TRAFFIC IS SLOWED, OVERLAY DIRECTION, MILES	1.0
53.2	DISTANCE TRAFFIC IS SLOWED, NON-OVERLAY DIR., MILES	0
53.3	PERCENT OF VEHICLES STOPPED, OVERLAY DIRECTION	0
53.4	PERCENT OF VEHICLES STOPPED, NON-OVERLAY DIRECTION	0
53.5	AVERAGE VEHICLE DELAY, OVERLAY DIRECTION, HRS	-0
53.6	AVERAGE VEHICLE DELAY, NON-OVERLAY DIRECTION, HRS	-0
54.1	ACP PRODUCTION RATE, CY/HR	70.
54.2	CRCP PRODUCTION RATE, CY/HR	70.
54.3	JCP PRODUCTION RATE, CY/HR	70.
54.4	BOND BREAKER PRODUCTION RATE, CY/HR	70.

DISTRESS/MAINTENANCE COST VARIABLES
****

55.1 DISTRESS REPAIR COST, CRCP OVERLAY, DOL 1000.00
55.2 INITIAL CRCP OVERLAY DISTRESS RATE, NO./MI/YR 1.0
55.3 SECONDARY CRCP OVERLAY DISTRESS RATE, NO./MI/YR 2.0
55.4 CRCP OVERLAY DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD-CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	3.0
2	5.0
3	8.0
4	10.0

56.1 DISTRESS REPAIR COST, JCP OVERLAY, DOL 1000.00
56.2 INITIAL JCP OVERLAY DISTRESS RATE, NO./MI/YR 1.0
56.3 SECONDARY JCP OVERLAY DISTRESS RATE, NO./MI/YR 2.0
56.4 JCP OVERLAY DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	4.0
2	6.0
3	9.0
4	18.0

57.1 DISTRESS REPAIR COST, ACP OVERLAY ON CRCP, DOL 500.00
57.2 INITIAL ACP/CRCP DISTRESS RATE, NO./MI/YR 1.2
57.3 SECONDARY ACP/CRCP DISTRESS RATE, NO./MI/YR 2.4
57.4 ACP/CRCP DISTRESS RATE FOR EACH YEAR AFTER LOSS
OF PAVEMENT LOAD-CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	3.6
2	6.0
3	9.0
4	18.0

58.1 DISTRESS REPAIR COST, ACP OVERLAY ON JCP, DOL 500.00
58.2 INITIAL ACP/JCP DISTRESS RATE, NO./MI/YR 2.0
58.3 SECONDARY ACP/JCP DISTRESS RATE, NO./MI/YR 4.0
58.4 ACP/JCP DISTRESS RATE FOR EACH YEAR AFTER LOSS
OF PAVEMENT LOAD CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	4.0
2	7.0
3	10.0
4	20.0

59.1 DISTRESS REPAIR COST, ACP OVERLAY ON ACP, DOL -0
59.2 INITIAL ACP/ACP DISTRESS RATE, NO./MI/YR -0
59.3 SECONDARY ACP/ACP DISTRESS RATE, NO./MI/YR -0
59.4 ACP/ACP DISTRESS RATE FOR EACH YEAR AFTER LOSS OF PAVEMENT LOAD CARRYING CAPACITY

YEAR AFTER FAILURE	DISTRESS RATE (NO./MILE)
1	-0
2	-0
3	-0
4	-0

COST RETURNS
****

60.1 SALVAGE VALUE, PERCENT OF OVERLAY CONSTRUCTION COST 10.0
60.2 VALUE OF EACH YEAR OF EXTENDED LIFE, DOL/SY/YR 0

COMBINED INTEREST AND INFLATION RATE
****

61.1 INTEREST RATE MINUS INFLATION RATE, PERCENT 5.0

APPENDIX C. REGRESSION MODELS FOR PREDICTING PAVEMENT RESPONSES

APPENDIX C. REGRESSION MODELS FOR PREDICTING PAVEMENT RESPONSES (Ref.4)

This appendix provides the four regression models used to develop the design charts for overlay thickness design.

The following information is provided for each regression model:

(1) an illustration of the pavement structure and the location of the predicted response,

(2) the details of the experiment used to generate the equation, i.e., the inference space over which the equation may be applied,

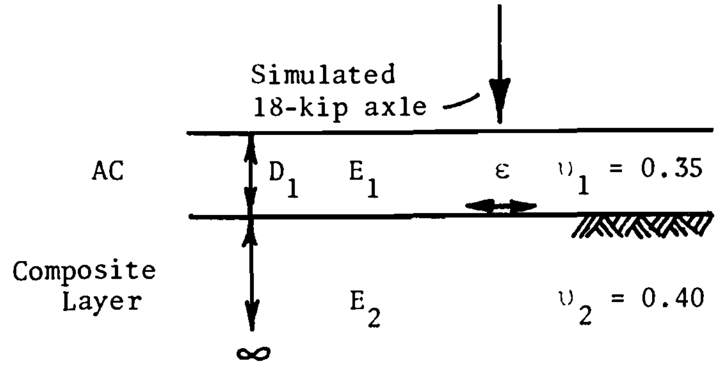
(3) the terms, coefficients and predictive accuracy of each equation, and

(4) an illustration (from the experimental data), of the predictive accuracy of the equation.

Pavement Structure

and Response:

(Code: E21)



Details of the Experiment:

1. Full factorial, 3 factors, no. of observations = $3^3 = 27$
2. Levels of the significant factors:

Factors	Levels: High	Medium	Low
E_1 (psi)	800,000	500,000	200,000
E_2 (psi)	60,000	40,000	20,000
D_1 (in)	10	7	4

Prediction Equation - E21:

$\log_{10} \sigma = \Sigma$ (Term x Coefficient)

$r^2 = 0.994$

std. error = 0.0204

Term	Coefficient
Intercept	$- 2.835 \times 10^{0}$
E_1	$- 9.309 \times 10^{-7}$
E_2	$- 7.530 \times 10^{-6}$
D_1	$- 5.870 \times 10^{-2}$
$(E_1)^2$	$+ 4.108 \times 10^{-13}$
$E_1 \times E_2$	$+ 4.035 \times 10^{-12}$
$E_1 \times D_1$	$- 2.356 \times 10^{-8}$

Fig C1. E21 Regression model: for predicting tensile strain in an asphalt concrete surface layer.

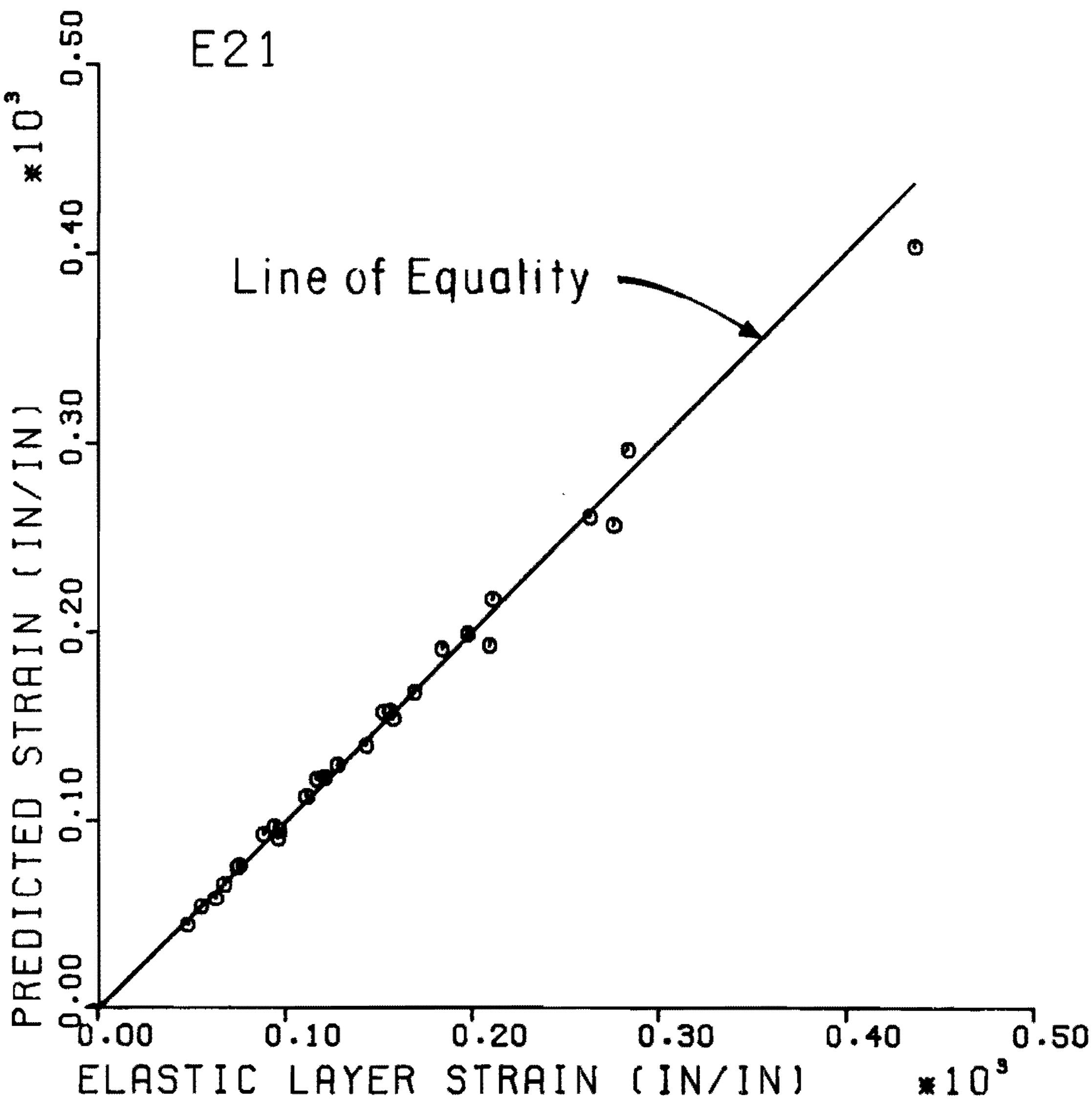
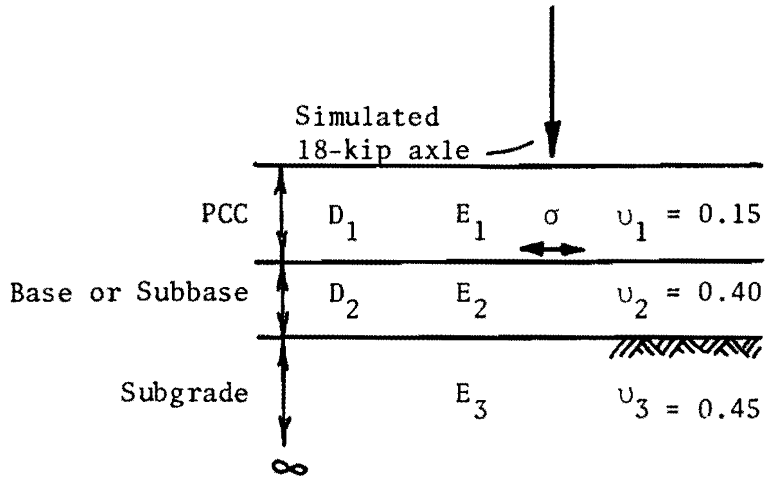


Fig C 2. Illustration of predictive accuracy of E21 Equation.

Pavement Structure

and Response:

(Code: S31)



Details of the Experiment:

1. Full factorial, 5 factors, no. of observations = 3^5 = 243
2. Levels of the significant factors:

Factors		Levels: High	Medium	Low
E_1	(psi)	6,500,000	5,000,000	3,500,000
E_2	(psi)	600,000	320,000	40,000
E_3	(psi)	20,000	11,000	2,000
D_1	(in)	10	8	6
D_2	(in)	12	9	6

(continued)

Fig C3. S31 Regression model: for predicting concrete stress in a 3-layer concrete pavement.

Prediction Equation - S31:

$\log_{10} \sigma = \Sigma$ (Term x Coefficient)

$r^2 = 0.989$

std. error = 0.0202

Term	Coefficient
Intercept	$+ 2.880 \times 10^{0}$
E_1	$+ 1.972 \times 10^{-8}$
E_2	$- 1.210 \times 10^{-6}$
E_3	$- 1.720 \times 10^{-5}$
D_1	$- 8.113 \times 10^{-2}$
$E_2 \times D_2$	$- 5.097 \times 10^{-8}$
$E_2 \times D_1$	$+ 6.121 \times 10^{-8}$
$E_1 \times E_2$	$+ 7.135 \times 10^{-14}$
$(E_3)^2$	$+ 3.818 \times 10^{-10}$
$(E_2)^2$	$+ 3.312 \times 10^{-13}$
$E_2 \times E_3$	$+ 5.034 \times 10^{-12}$

Fig C3. (continued)

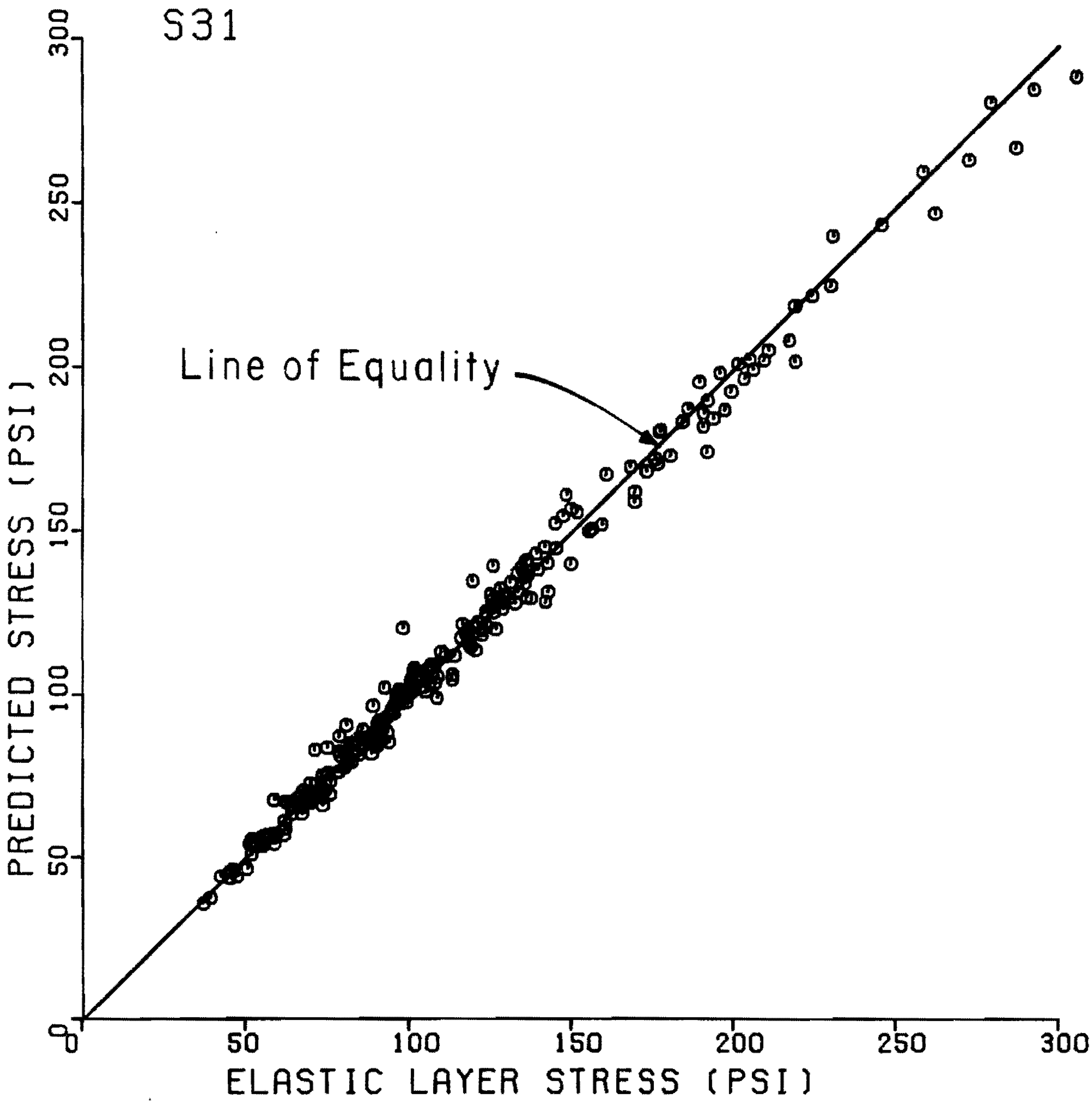
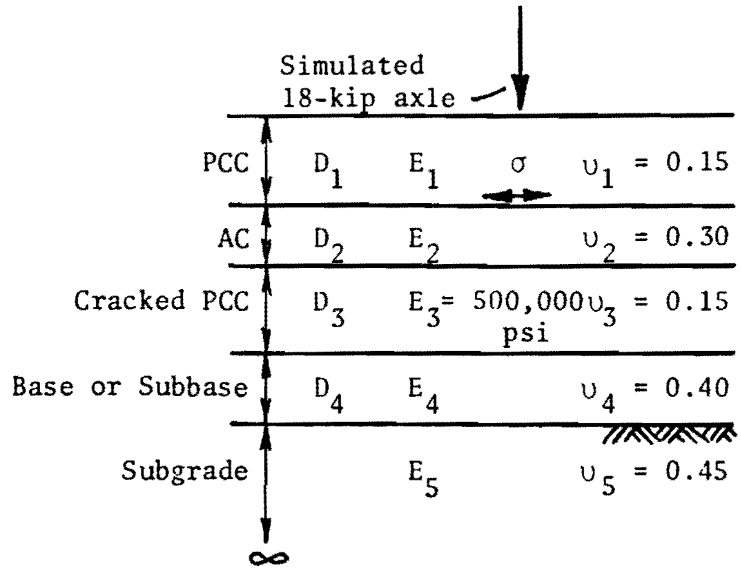


Fig C 4. Illustration of predictive accuracy of S31 equation.

Pavement Structure

and Response:

(Code: SU51C)



Details of the Experiment:

1. Fractional factorial, 8 factors, no. of observations = $\frac{1}{27} \times 3^8 = 243$
2. Levels of the significant factors:

Factors		Levels High	Medium	Low
E_1	(psi)	6,500,000	5,000,000	3,500,000
E_2	(psi)	550,000	300,000	50,000
E_4	(psi)	500,000	270,000	40,000
E_5	(psi)	20,000	11,000	2,000
D_1	(in)	8	7	6
D_2	(in)	7	4	1
D_3	(in)	10	8	6
D_4	(in)	12	9	6

Note: E_3 in experiment was fixed at 500,000 psi to simulate cracked PCC.

(continued)

Fig C.5. SU51C Regression Model: for predicting stress in unbonded concrete overlay of 5-layer concrete pavement (original PCC cracked).

Prediction Equation - SU51C:

$\log_{10} \sigma = \Sigma$ (Term x Coefficient)

$r^2 = 0.986$

std. error = 0.0189

Term	Coefficient
Intercept	$+ 2.696 \times 10^{0}$
E_1	$+ 3.354 \times 10^{-8}$
E_2	$- 1.455 \times 10^{-6}$
E_4	$- 5.530 \times 10^{-7}$
E_5	$- 4.660 \times 10^{-6}$
D_1	$- 7.299 \times 10^{-2}$
D_3	$- 2.170 \times 10^{-2}$
D_4	$- 1.899 \times 10^{-3}$
$E_2 \times D_2$	$- 5.900 \times 10^{-8}$
$(E_2)^2$	$+ 8.141 \times 10^{-13}$
$E_1 \times E_2$	$+ 6.601 \times 10^{-14}$
$E_4 \times D_2$	$+ 2.420 \times 10^{-8}$
$E_2 \times D_1$	$+ 5.436 \times 10^{-8}$
$(E_4)^2$	$+ 3.316 \times 10^{-13}$
$E_4 \times E_5$	$+ 5.807 \times 10^{-12}$
$D_2 \times D_3$	$+ 1.066 \times 10^{-3}$
$E_4 \times D_4$	$- 1.267 \times 10^{-8}$
$E_4 \times D_3$	$+ 1.869 \times 10^{-8}$
$E_5 \times D_2$	$+ 2.859 \times 10^{-7}$
$E_2 \times E_4$	$- 1.273 \times 10^{-13}$

Fig C5. (Continued)

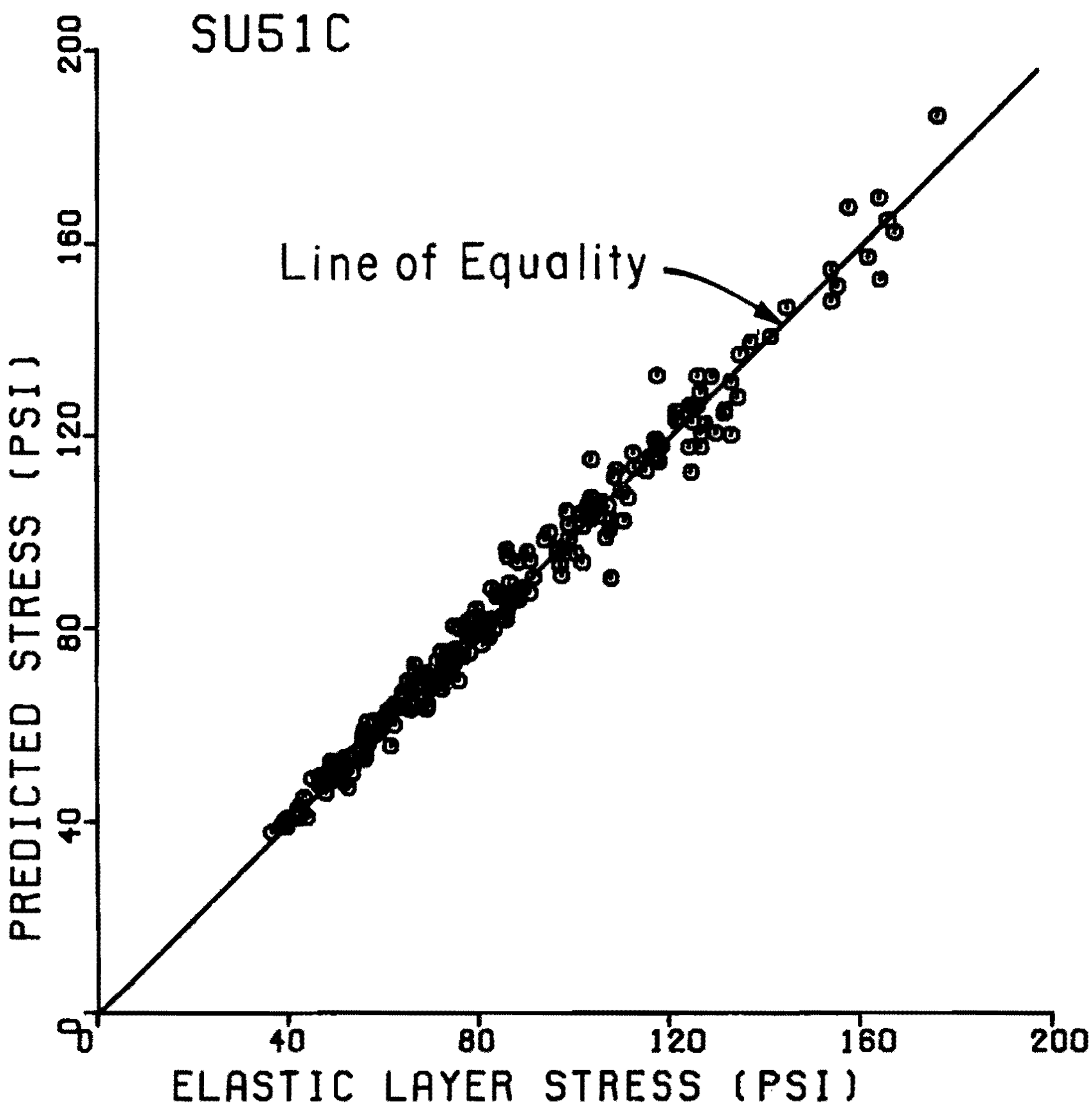
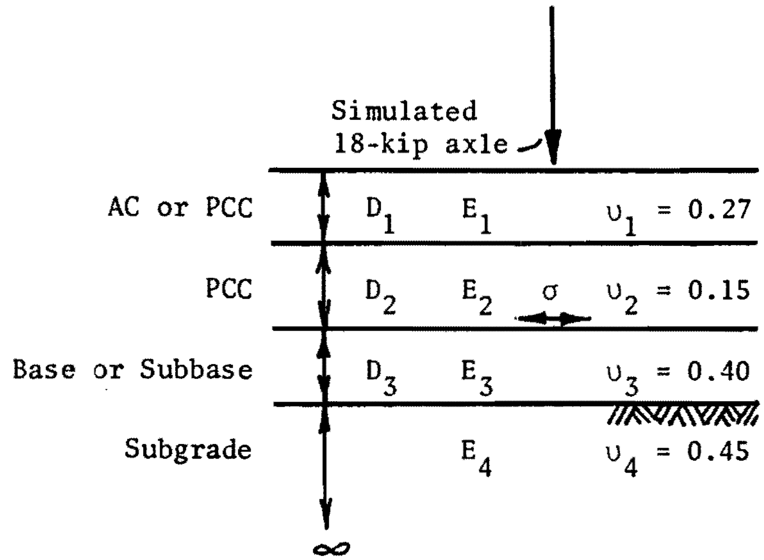


Fig C6. Illustration of predictive accuracy of SU51C equation.

Pavement Structure
and Response:

(Code: SB42)



Details of the Experiment:

1. Fractional factorial, 7 factors, no. of observations = $\frac{1}{9} \times 3^7 = 243$
2. Levels of the significant factors:

Factors		Levels High	Medium	Low
E_1	(psi)	6,250,000	3,250,000	250,000
E_2	(psi)	6,500,000	5,000,000	3,500,000
E_3	(psi)	600,000	320,000	40,000
E_4	(psi)	20,000	11,000	2,000
D_1	(in)	8	6.5	5
D_2	(in)	10	8	6
D_3	(in)	12	9	6

(continued)

Fig C7 SB42 Regression Model: for predicting stress in a 5-layer concrete pavement with a bonded overlay.

Prediction Equation - SB42:

$$\log_{10} \sigma = \Sigma(\text{Term x Coefficient})$$

$r^2 = 0.987$

std. error = 0.0213

Term	Coefficient
Intercept	$+ 2.667 \times 10^{0}$
E_1	$- 9.509 \times 10^{-8}$
E_2	$+ 3.368 \times 10^{-8}$
E_3	$- 6.977 \times 10^{-7}$
E_4	$- 1.644 \times 10^{-5}$
D_1	$- 2.719 \times 10^{-2}$
D_2	$- 5.441 \times 10^{-2}$
D_3	$- 5.797 \times 10^{-3}$
$(E_1)^2$	$+ 8.702 \times 10^{-15}$
$E_3 \times D_3$	$- 3.814 \times 10^{-8}$
$E_1 \times E_3$	$+ 9.055 \times 10^{-14}$
$(E_4)^2$	$+ 3.764 \times 10^{-10}$
$E_2 \times E_3$	$+ 4.602 \times 10^{-14}$
$E_3 \times D_2$	$+ 3.225 \times 10^{-8}$
$E_1 \times D_1$	$- 2.891 \times 10^{-9}$
$E_1 \times D_3$	$+ 1.323 \times 10^{-9}$
$E_3 \times (E_1)^2$	$- 8.115 \times 10^{-21}$

Fig C7 (Continued)

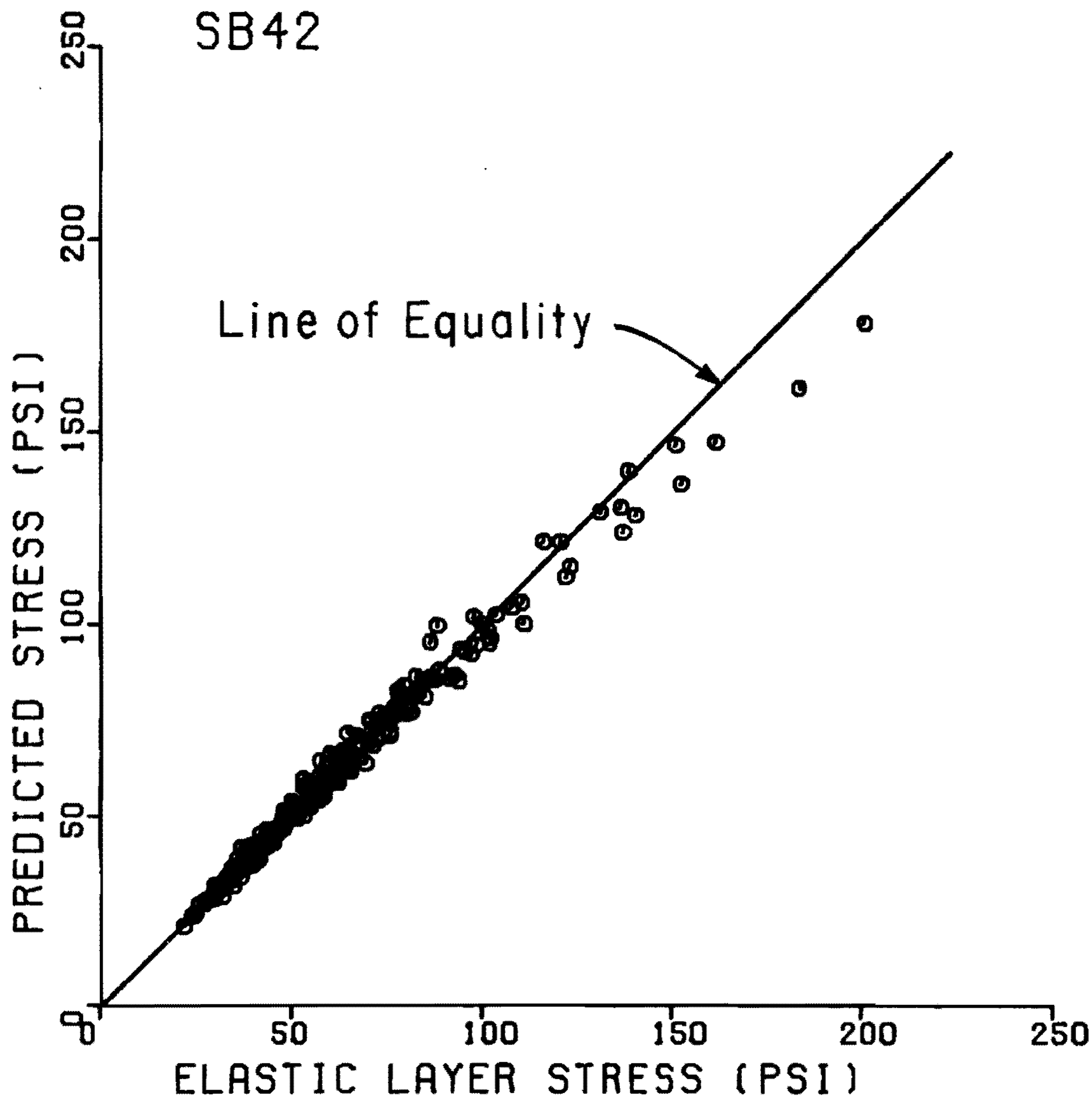


Fig C8 Illustration of predictive accuracy of SB42 equation.